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TRANSACTIONS

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OF

CIVIL ENGINEERS

(INSTITUTED 1852)

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ERRATA

Transactions, Vol. LXXIX.

Page 1435, line 8, for "William Jacomb-Hood", read "William Jacomb".

Transactions, Vol. LXXX.

Page 1495, line 8, insert the words "twice the", before "live-load", making the line read: "shall be proportioned for the algebraic sum of the dead- and twice the live-load".

Page 1496, line 8, for "16.54 sq. in.", read "16.64 sq. in.".

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

TRANSACTIONS

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Paper No. 1352

WATER SUPPLY OF THE SAN FRANCISCO-OAKLAND METROPOLITAN DISTRICT*

By H. T. Cory, M. Am. Soc. C. E.

With Discussion by Messrs. F. T. Robson, L. J. Le Conte, C. E. Grunsky, William Schuyler Post, M. M. O'Shaughnessy, Allen Hazen, J. D. Galloway, F. G. Baum, E. G. Hopson, O. W. Peterson, Carl J. Rhodin, Clement H. Miller, Rudolph W. Van Norden, Burton Smith, C. E. Sloan, Edwin Duryea, Jr., H. H. Wadsworth, W. C. Hammatt, A. Griffin, J. Horace McFarland, F. C. Herrmann, Walter Henry Brown, and H. T. Cory.

Synopsis.

This paper deals particularly with the municipal water supplies, present and future, of the cities around the Bay of San Francisco, and incidentally of the so-called Hetch Hetchy project, for which Congress recently granted the necessary permits.

First, a detailed study is made of the population growth of the San Francisco-Oakland Metropolitan District and its several subdivisions. The conclusions reached are that, probably:

The East Bay portions will pass San Francisco proper in population, and to a greater extent in water requirements, in about 20 years;

The next census will show the entire district second to Greater Los Angeles; and that,

* Presented at the meeting of November 4th, 1914.

After a normal growth for two or three decades, Greater San Francisco-Oakland, and especially the East Bay district, will, for a few decades, grow very rapidly.

Then follows a brief description of existing systems and statements of their possible full development.

The question as to whether the Bay cities need and can afford a distant supply in the immediate future is then discussed.

Four or five decades hence, with the utmost co-operation of all interests, local supplies must be supplemented by water brought in from a distance.

The last two parts deal, respectively, with the considerations governing a wise choice of such distant supply or supplies, and the various possibilities.

The conclusions are that the problem is much more, and almost exclusively, one for the East Bay cities, than for San Francisco, and the desirability of considering the situation from this standpoint very carefully, before definitely fixing on the Hetch Hetchy project, is advised.

Introduction.

Urban communities on San Francisco Bay in California have proposed to secure additional water supplies. After more than a decade of endeavor, the City of San Francisco, acting in its own name, but latterly in behalf of a proposed metropolitan water district, has just secured permission to use the Hetch Hetchy Valley and dam site on the main Tuolumne River, within the boundaries of the Yosemite National Park. The privilege of doing this was bitterly contested by numerous individuals, societies, and associations. Widespread interest was aroused, and much feeling engendered.

Because of the interesting nature of the problem to the Profession; because of the many engineering facts which have been brought out from time to time in a multitude of generally inaccessible reports and hearings before Government officials and Congresses; and because various engineers have differed so widely in regard to many matters, a comprehensive paper on the subject is believed to be timely.

Topography of San Francisco Metropolitan Area.—San Francisco Bay, with its outlet to the ocean—the Golden Gate—is about in the middle of the California coast line.

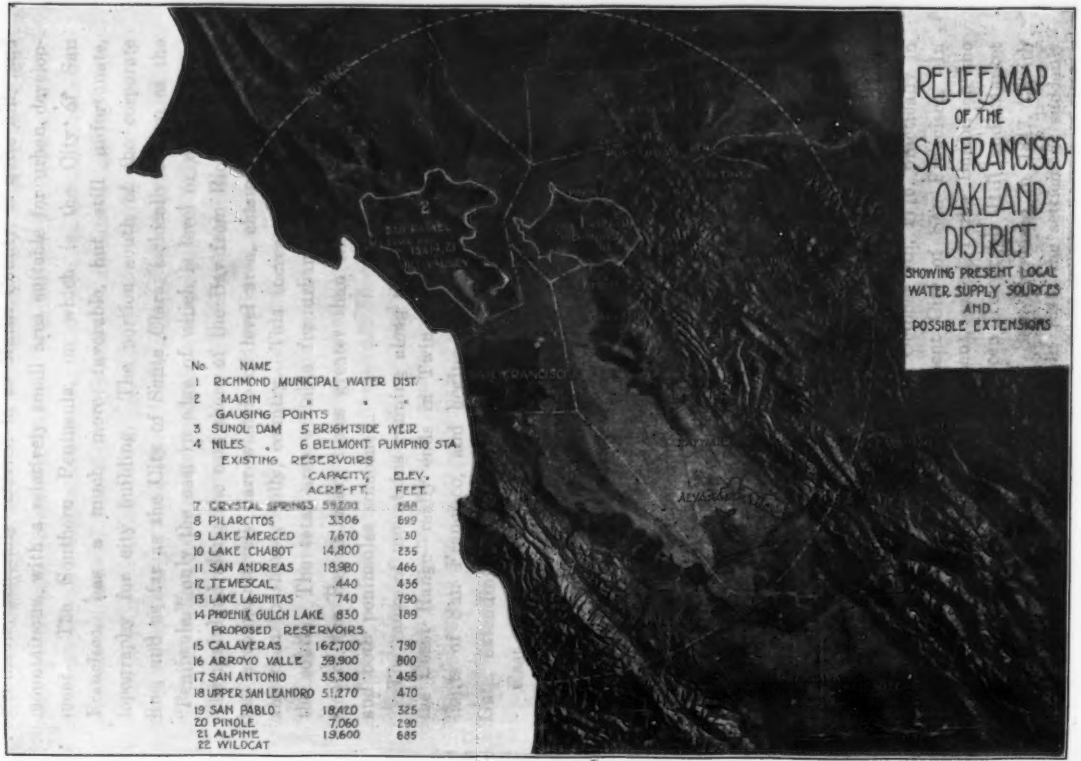
RELIEF MAP
OF THE
SAN FRANCISCO-
OAKLAND
DISTRICT
SHOWING PRESENT LOCAL
WATER SUPPLY SOURCES
AND
POSSIBLE EXTENSIONS
No. NAME
1 RICHMOND MUNICIPAL WATER DIST.
2 MARIN " "
GAUGING POINTS
3 SUNOL DAM 5 BRIGHTSIDE WEIR
4 NILES " 6 BELMONT PUMPING STA.
EXISTING RESERVOIRS
CAPACITY ACRE-FT. ELEV. FEET
7 CRYSTAL SPRINGS 59,200 288
8 PILARCITOS 3306 699
9 LAKE MERCED 7,670 30
10 LAKE CHABOT 14,800 235
11 SAN ANDREAS 18,980 466
12 TEMESCAL 440 436
13 LAKE LAGUNITAS 740 790
14 PHOENIX GULCH LAKE 830 189
PROPOSED RESERVOIRS
15 CALAVERAS 162,700 730
16 ARROYO VALLE 39,900 800
17 SAN ANTONIO 35,300 455
18 UPPER SAN LEANDRO 51,270 470
19 SAN PABLO 18,420 325
20 PINOLE 7,060 290
21 ALPINE 19,600 685
22 WILDCAT

FIG. 1.

The relief map, Fig. 1, shows the immediate region. The Northern Peninsula, generally referred to as Marin County, is quite hilly and mountainous, with a relatively small area suitable for urban development. The Southern Peninsula, on which is the City of San Francisco, has a much more favorable, but still unfortunate, topography for city building. The portion south of the corporate line, and as far as the City of Santa Clara, is locally known as the "Peninsula," only the eastern edge of which is level or relatively so. The mainland along the east shore of the Bay from Richmond south has a comparatively narrow strip of level area, changing to rolling, and then to sharply hilly, country. This increases in width toward the south. The total area suitable for urban development on this side of the Bay is many times greater than that in San Francisco and both peninsulas combined.

The range of mountains running along the California coast line—the Coast Range—really ends in Twin Peaks, within the corporate limits of San Francisco, and begins again just across the Golden Gate, extending thence northward.

East of the Bay, another mountain range, parallel to the Coast Range but higher and wider, extends across the entire region, with a very narrow break, known as Carquinez Straits, through which the combined waters of the Sacramento and San Joaquin Rivers are discharged into San Francisco Bay. In many ways this Mt. Diablo Range is a most important topographical feature of the San Francisco Metropolitan District. It modifies in a striking way the local climatic conditions; stands as a barrier against urban growth eastward; affords productive water-sheds and storage facilities for local water developments; and is a very serious obstacle to be overcome in bringing a water supply to the Bay region from those sections of the Sierra Nevada Mountains directly to the east.

Urban Development.—The Mission of San Francisco was established by the Franciscan Fathers on October 9th, 1776, practically in the geographical center of the present City of San Francisco. In 1846 there was only a little settlement, called Yerba Buena, on the shores of Yerba Buena Cove, between Telegraph Hill and Market Street. The name was changed to San Francisco in 1847. With the discovery of gold in California in 1848, the settlement suddenly

became important. For a few decades the growth was phenomenal and then dropped to about the normal in cities of its size.

It is unfortunate that the original settlement was not on the mainland, as the available area for satisfactory city development on the Southern Peninsula is relatively small, and is sharply restricted by the topography.

Effect of Topography.—The effect of the local topography has been very marked. The disposition of the land masses affects the fog distribution and results in a peculiarly great variation in climatic conditions over the district. San Francisco's climate is foggy and windy, generally speaking, with summer temperatures only slightly higher than those of winter, the thermometer rarely reaching 80° Fahr., and even a few scattering snowflakes and thin films of ice are to be seen only at intervals of years.*

The city is compactly built, due to the limited area which is not on steep hillsides. As the climate is really quite equable, it is unfortunate that home building was not carried out along the same lines as in Los Angeles and all other California cities, namely, to accentuate and take advantage of the famous climate.

In 1874, the area was materially enlarged by the introduction of cable cars, which were developed to meet the local situation. A railroad built into the interior of the State in September, 1869, naturally ended on the east shore of the Bay, with ferry service into the city proper. The suburban service thus available to the mainland, started settlements in Alameda, Oakland, and Berkeley, where ample space, milder climatic conditions, and cheap individual water supplies resulted in a different type of city building.

At first ocean shipping was of pre-eminent importance, and the development was determined by deep-water wharfage. As railway transportation increased in relative importance, commerce was influenced by the railway terminals on the mainland, but the position of San Francisco as the commercial center has become so firmly established that it is doubtful whether it will ever be transferred across the Bay.

Effect of Geographical Location.—In the nature of the case, San Francisco's population is cosmopolitan and self-sufficient. The city very quickly became a metropolis, and was effectually isolated from

* The last snowfall was 1 in., on March 3d, 1896.

any other center. It was the financial and commercial hub of western gold mining enterprise, which attracted the energetic and adventurous from all over the world. There are large foreign quarters in other cities of the country, but in San Francisco—except in Chinatown—the many nationalities are diffused. Consequently, its civic, religious, educational, and political ideas and institutions were and are different from those obtaining anywhere else in the United States. It is financially self-made, self-centered, and self-sufficient. The prevailing spirit was, and in large measure is yet, against San Francisco being a "cheap town." It is a wealthy one. Large personal freedom developed, and precedents of all kinds, including engineering methods, quickly came to be regarded lightly—often with excellent results.

In consequence, individualism was developed to an extreme, with a corresponding lack of co-operation. This condition obtains in a less, but still marked, degree to-day.

Considerable manufacturing necessarily developed, and, until the last decade, little was done on the Pacific Coast outside of the city. It became the center of Labor Unionism on the Coast, more particularly after the successful protests of Dennis Kearney and his followers against Asiatic labor competition.

Limits of Greater San Francisco-Oakland, as Used in This Paper.—The census adopts for metropolitan districts all urban population within 10 miles of the municipal boundaries. In this paper, however, the term will be used as including the territory which, logically, should ultimately have a common water supply. It is practically coincident with the region circumscribed by a circle of 30 miles radius with the San Francisco Ferry Building as the center. Greater Los Angeles, similarly, is considered as the area within a radius of 30 miles from the Los Angeles City Hall.

I.—Probable Future Population.

Growth of San Francisco.—For the past three decades the growth of the San Francisco Metropolitan District has been steady, and at the average rate for American cities of like population, but very much slower than Chicago, New York, and several others, when they were of similar size. The disastrous conflagration following the earthquake of April, 1906, temporarily affected San Francisco proper most seriously, but markedly accelerated the development of the East Bay

cities and other suburbs. Such effect was not great enough to explain the relatively slow growth of Greater San Francisco-Oakland as compared with all other important centers on the Pacific Coast and in the Far West during recent years.

The reasons for this, and how potent they will be in the future, are questions of primal importance. The population, and consequently the water requirements, are the first things to be determined in planning a water supply.

So far as the writer is aware, those who have estimated the future population of the district have not analyzed the peculiar conditions in California. The phenomenal percentages of increase in Los Angeles, Cal., Vancouver, B. C., Seattle, Wash., and other Pacific Coast cities in recent years, have never been taken into account, except by adding a statement that, remembering them, "the curve may be regarded as extremely conservative." This is not very illuminating.

Growth of California.—In the decade, 1900 to 1910, California's population increased 60.1 per cent. The rate for Oregon was 62.7%, Nevada, 93.4%, Idaho, 101.3%, and Washington, 120.4 per cent. The weighted average for these four other States was 95.9%, and for Washington and Oregon—the two other Pacific Coast States—95.8 per cent. The numerical increase was, Washington, 623 867; Oregon, 259 229; or together, 883 116—only 11 382 less than California.

Relative Growth of Northern and Southern California.—On examining detailed figures, it is found that the excessive population increase of the State is not of California generally, but of Southern California, consisting of the seven counties south of the Tehachapi—Ventura, Los Angeles, San Bernardino, Orange, Riverside, San Diego, and Imperial. The growth of population in these seven counties was 154%; in the rest of the State it was only 38%, and in the census San Francisco Metropolitan District but 41 per cent. The curves, Figs. 2 to 10, indicate that, unless something in the nature of widespread calamity or war occurs to change the respective rates of growth, Southern California will, by 1920, comprise more than half the State in all essential respects except area.

There are several reasons why this is surprising. They must in time be effective. These, in the probable order of importance, are:

1.—The natural resources of the State, like its precipitation, increase approximately in direct ratio from south to north. A marked exception, whereby the reverse is true, is the attractiveness of the ocean beaches.

2.—The climatic conditions of Southern California, from a residence point of view, are inferior to those of the central and northern

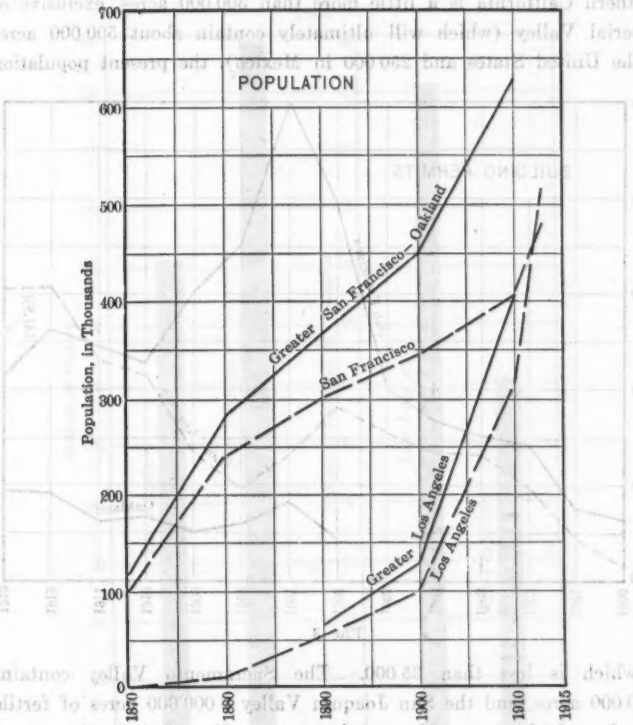

FIG. 2.

parts. This fact has been obscured by the effective advertising of Southern California. Greater San Francisco-Oakland is climatically a markedly better summer resort than Los Angeles, and probably is not equalled anywhere in America.

3.—Within easy reach of San Francisco lies some of the most picturesque scenery in the world. There are opportunities to con-

struct about the Bay cities the most attractive and beautiful roads and boulevards on the Continent, and the Bay is almost unrivaled for yachting and motor boating the year round.

4.—The agricultural and horticultural possibilities of Southern California are literally dwarfed by those of the Central Valley—Sacramento and San Joaquin Valleys—alone. The total irrigated area of Southern California is a little more than 300 000 acres, exclusive of Imperial Valley (which will ultimately contain about 500 000 acres in the United States and 250 000 in Mexico), the present population

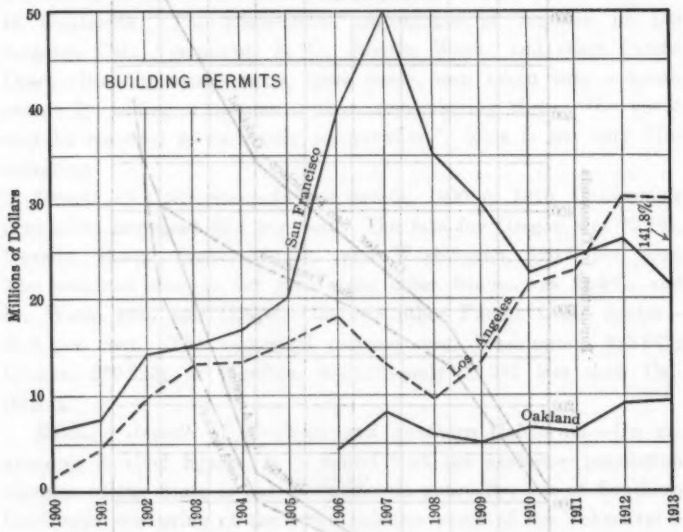

FIG. 3.

of which is less than 35 000. The Sacramento Valley contains 2 250 000 acres, and the San Joaquin Valley 5 000 000 acres of fertile land for which thoroughly satisfactory supplies of irrigation water may be developed at a cost per acre considered low in Southern California. That is, the agricultural development of all Southern California, which was the foundation for its entire development, is only 4% of the easily irrigable area of the Central Valley alone. Furthermore, the proportion of irrigable area adapted to growing oranges, lemons, grape-fruit, etc., is about the same in each region. When it is

remembered that California produces 40% of the world's commercially grown oranges, and that the groves scattered about Sacramento and San Joaquin Valleys have shown that oranges ripen there at least 6 weeks earlier than in Southern California, and suffer less from frosts, some realization of the situation can be obtained.

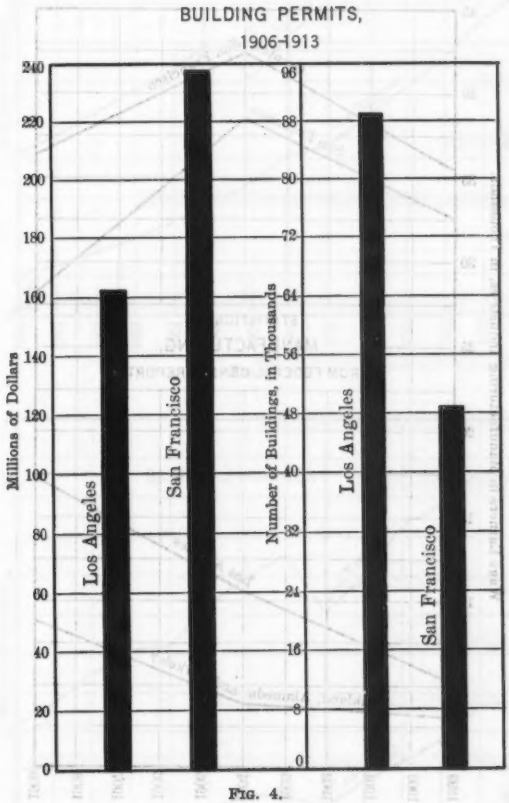

FIG. 4.

The total area of irrigable land logically tributary to San Francisco is 12 500 000 acres, and that of arable land is 15 393 000 acres.

5.—Thirty years ago, when Los Angeles had only 12 000 inhabitants—mostly Mexicans—San Francisco had 250 000, and was the "City" to which people "went down" from a vast region extending

nearly 1 000 miles south, east, and north; it was practically the only seaport on the Pacific Coast, and for more than 10 years was the western terminus of the only transcontinental railway.

In other words, the soil of Southern California is, to say the least, no more fertile; the water supply is by comparison pitifully

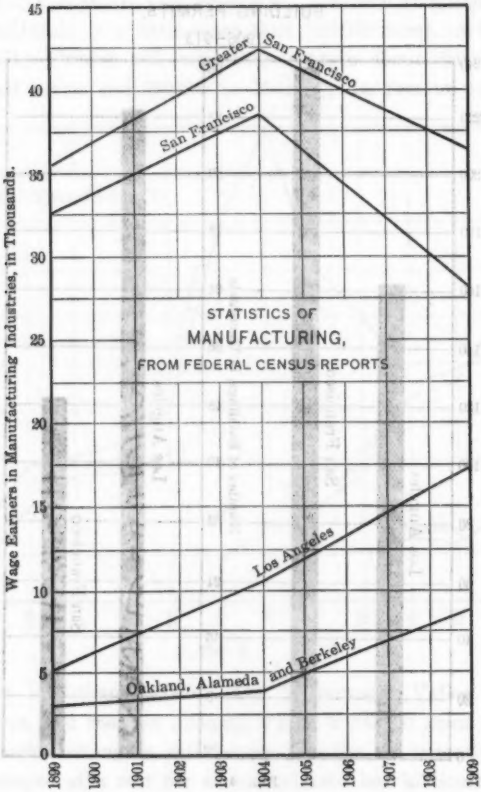

FIG. 5.

deficient; the climate not so delightful nor perfectly adapted to fruit culture, especially oranges and lemons; the extent of back country which can ever be made suitable for intensive farming, but a small fraction of that logically tributary to San Francisco; the natural

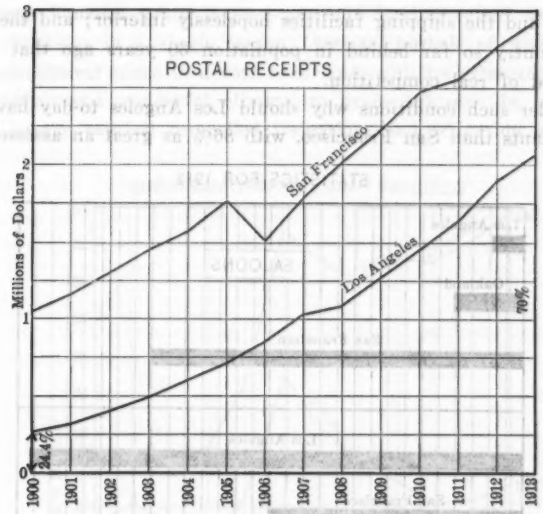
POSTAL RECEIPTS
Millions of Dollars
3
2
1
0
San Francisco
Los Angeles
70%
34.4%
1900
1901
1902
1903
1904
1905
1906
1907
1908
1909
1910
1911
1912
1913

FIG. 6.

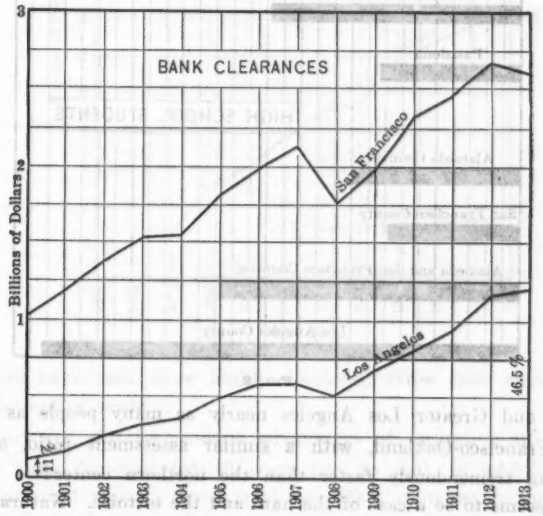
BANK CLEARANCES
Billions of Dollars
3
2
1
0
San Francisco
Los Angeles
46.5%
1900
1901
1902
1903
1904
1905
1906
1907
1908
1909
1910
1911
1912
1913

FIG. 7.

harbor and the shipping facilities hopelessly inferior; and the southern country so far behind in population 30 years ago that no one dreamed of real competition.

Under such conditions why should Los Angeles to-day have more inhabitants than San Francisco, with 86% as great an assessed valu-

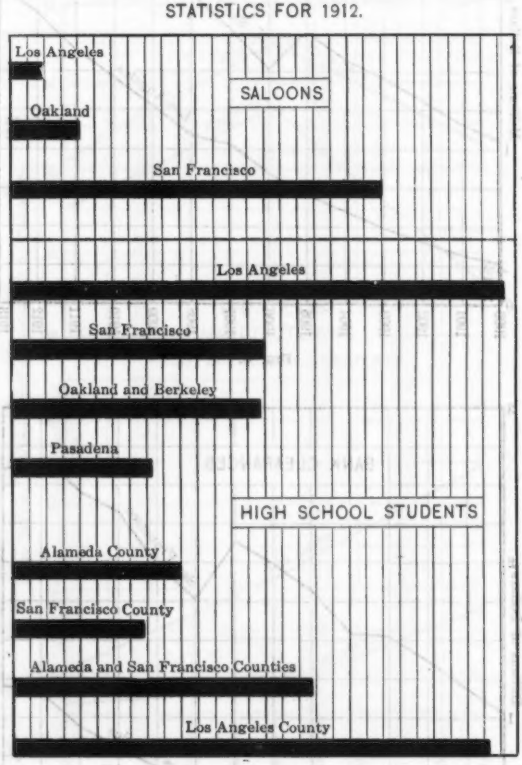

FIG. 8.

ation, and Greater Los Angeles nearly as many people as Greater San Francisco-Oakland, with a similar assessment ratio, and still growing tremendously faster than the northern center?

It seems to be a case of the hare and the tortoise. Natural advantages in the Sacramento and Upper San Joaquin Valleys resulted

in great bonanza grain farms—one man having raised on his own land holdings in a single season 1 000 000 bushels of wheat. It is an agricultural crime to devote such land, with its extraordinary crop adaptation and wonderful climate, to grain culture. In the Southland scanty rainfall compelled irrigation, and irrigation and citrus

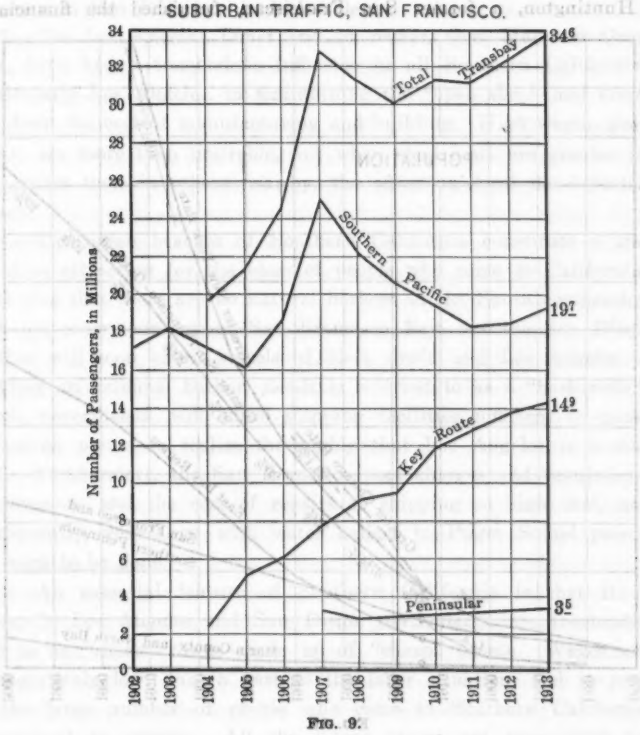

FIG. 9.

culture have been more highly developed there than anywhere else in the entire world.

However, the tremendous advantages of irrigation in the State were fully demonstrated 40 years ago, so that, in addition to this fundamental reason, there must be others quite as potent. These, in the probable order of importance, are:

1.—The Los Angeles Chamber of Commerce. This is doubtless the most efficient organization of its kind in the world, presenting a united, brave front in all matters affecting the development of, not only Los Angeles, but all Southern California. It is doubtless unequalled as an effective advertiser.

2.—The Pacific Electric Interurban System, for which Mr. H. E. Huntington, a former San Franciscan, furnished the financial

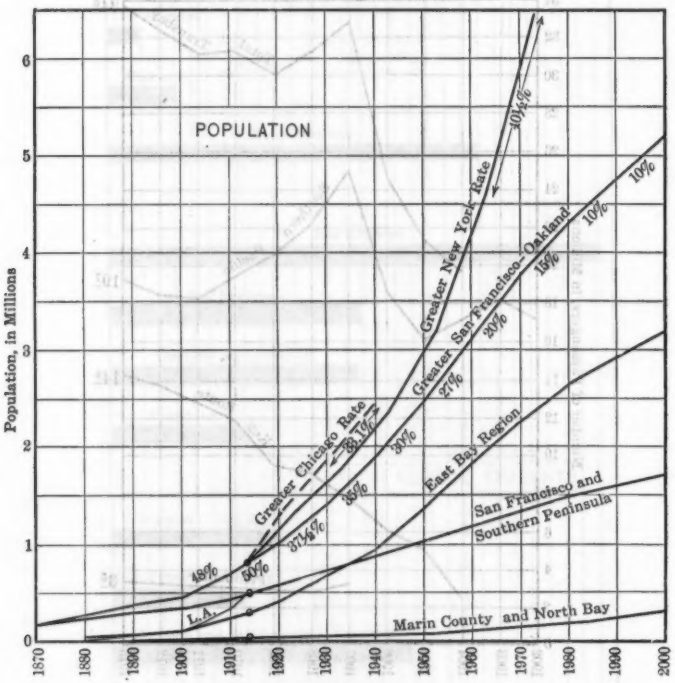

FIG. 10.

strength, and Mr. Epes Randolph, now of Tucson, Ariz., outlined the broad general plans. It is the most complete, extensive, well-constructed, and well-equipped system of interurban electric railroads in the world. There are 6 200 daily train movements over 47 routes, the total number of passengers carried yearly being more than 106 000 000. The total annual trans-bay suburban traffic between San

Francisco and Oakland, Berkeley, etc., is less than 35 000 000, or about one-third, in spite of the fact that such suburban service is, next to that of Los Angeles, the best and cheapest on the Continent.

In addition, there is doubtless no city in the United States—and that means the world—with a system of surface electric roads as modern and well-equipped as Los Angeles. The street railway situation in San Francisco is deplorable.

3.—The *Los Angeles Times* and its owner, Gen. Harrison Gray Otis, have had a tremendous influence in all Southern California, particularly Los Angeles, in maintaining the "open shop" and keeping down the cost of manufacturing and building. High wages, generally, are more than desirable, but when shop costs are greater in one center than in others near-by, the effect on local development is bad.

4.—The ocean beaches of Southern California constitute a tremendous attraction for the class of people who come to California. It is true that there are no natural harbors in the South comparable with the marvelous one of San Francisco Bay, but the San Diego Harbor will soon admit vessels of 35-ft. draft, and Los Angeles is building an artificial harbor, jocularly referred to as a "harborette," which, nevertheless, will afford shipping facilities sufficient to cause competing points to realize thoroughly that Los Angeles is a seaport. Furthermore, the San Francisco port charges and regulations are onerous, and the cost of repairs to shipping so high that, not infrequently, vessels go with water ballast to Puget Sound points for work to be done.

5.—An essential feature of Southern California is that it is "cheap." Los Angeles and San Diego, particularly, are frequently said to be "cheap" towns made up of "cheap" people. Wages are comparatively low, due in part to the labor situation, and in part to the large number of people who come to Southern California determined to remain. All the larger places are surrounded by country topographically suited to indefinite urban expansion in all directions. Consequently, labor and rents are cheap. The expression "Southern Cafeteria," used derisively, has played a very important part.

6.—The fact that Southern Californians are often sneeringly referred to as "goo-goos" (goody-goody) has been an effective adver-

tisement. San Francisco, on the other hand, has been widely advertised as the "Paris of America." There are 200 saloons in Los Angeles and 2 200 in San Francisco proper, and Los Angeles has more than twice as many high school students.

7.—Until a decade ago, San Francisco was, politically, the State of California, and was practically governed by railway interests. Of all boss-ridden commonwealths in the United States, California took first rank, and bribery, "influence," and manipulation of public affairs for the Interests controlling the machine became accepted as a matter of course. The moral fiber of the community was seriously affected. The influx into San Francisco was relatively small, but the tremendous increase in the population of Los Angeles came from all over the United States, bringing a strong strain of Puritanism and higher civic standards. The result for Los Angeles is a community which can and does co-operate and trust its leaders, and secures vastly more advanced civic improvements, of which the water supply is the most striking.

The years of wrangling about San Francisco's water supply and private ownership of the present water system are not accidents, they are only some of the costs of its individualistic temperament.

8.—Because the newcomers quickly outnumbered the natives, Southern California is really not "Western" at all, but a sort of general average of the entire United States and Canada—particularly east of the Rocky Mountains—in education, political and religious ideals, institutions, and spirit. The large cost of railway transportation from New York to the Coast has kept the percentage of European immigration very low. The influx into the State naturally goes to that portion having the same civilization and ideals as the newcomers. Consequently, almost everybody in America has acquaintances, friends, or relatives in Southern California—often very many of them. The result is an "endless chain" never elsewhere equalled.

9.—This factor has been intensified by the limited areas available for intensive cultivation. No large irrigation or colonization schemes were there desirable or even possible. Small, entirely distinct and independent tracts were often settled almost wholly by people from one general community. For example, the Ontario Project, promoted in 1888 and carried to a successful conclusion by

Messrs. William and George Chaffey, formerly of Ontario, Canada, was largely settled by Canadians from that Province, and hence the name. The various settlements—Riverside, Redlands, San Bernardino, Ontario, Pomona, etc.—have retained their individuality. All newcomers have been pouring into small tracts having a gross area of about 300 000 acres, so that the real population density of the inhabited portions of the region is fairly high. The communities have been well established since 1895, and with practically no pioneering work since 1900. So far as municipalities are concerned, however, the reverse is true, in that there are unlimited areas available for urban growth in every direction.

10.—Due to the progressive spirit, the high average and extremely well distributed wealth, and the fact that the country is a tourist playground, automobiling in Southern California is more general than in any part of the world. This quickly resulted in the construction throughout almost the entire region of a system of beautiful boulevards. These have gone far to eliminate what little "pioneering" work was left to be done there a few years ago.

11.—All the scenic attractions of the Southland have been made easy and comfortable of access and brought emphatically to the attention of every tourist. Although the San Francisco Bay region naturally offers much more in every way except ocean bathing, and might be made one of California's greatest scenic assets, this certainly has not yet been done. Thus, Southern California has come to be regarded as the nation's playground and residence section *de luxe*, where the tourist spends his weeks and months while his visit to San Francisco is ordinarily of but a few days *en route*. For this reason it is extremely probable that the Panama-Pacific Exposition in San Francisco in 1915 will really benefit Southern California much more than the Bay region, especially as another peculiarly unique and attractive, though less expensive, Exposition will be held simultaneously in San Diego.

12.—California is in an earthquake region in the sense that *tremblors* are not infrequent. The damage from the shocks of April 18th, 1906, in the East Bay cities was insignificant. In San Francisco proper the chief damage was the breaking of the gas and water pipes in the filled-in areas, thus interrupting the main water supply. This caused one of the disastrous fires of history.

Property worth $350 000 000 was wiped out of existence, many of the inhabitants were widely scattered, and the normal development of the city was violently interrupted. Los Angeles, and to a much less degree, other Coast cities, gained thereby. The final effect on the size of the two cities, however, has been much less than is generally supposed. San Franciscans have nearly all returned. The financial status of many individuals was greatly altered, and the city as a whole is less wealthy than it would have been. Otherwise it, and particularly Greater San Francisco-Oakland, was benefited in many important ways. The effect of the disaster on the location of newcomers to the State was very marked for a time, and doubtless is still appreciable. However, even a great disaster rapidly fades from the world's memory. Los Angeles, quite as much as San Francisco, is in the *tremblor* district, and has fully as many tremors, and by the law of averages the northern city is the safer. Nowhere in the State has a really severe earthquake occurred in historic times; disastrous shocks are like lightning, in that they may strike anywhere. These facts are now generally understood and realized.

Class of Domestic Immigrants.—The people who come to California have not the slightest intention of pioneering, but rather of obtaining the pleasantest home within their means. This is a fundamental fact. That agricultural and suburban land values are from two to five times higher in Southern California than in the Northern sections has not yet had any marked effect, although, of course, it must in time.

It is of the greatest importance to realize that, 20 years ago, when the growth of Southern California began to be really phenomenal—not so much in percentage of population increase as in actual amount and character—the region was agriculturally and horticulturally quite fully developed. There were thrifty, well-matured groves and orchards covering more than two-thirds of the area now devoted to horticulture, with practically all the present steam railway mileage in operation, and beach, park, driveway, and scenic resort development done. In short, all "pioneering" was a thing of the past.

It was then that the future began to be discounted, boosting became general, the Pacific Electric System was built, the oil fields were exploited, and extraordinary growth commenced.

Probable Future Growth Within the State and of Greater San Francisco-Oakland.—To what extent and how quickly may these influences and conditions be expected to change, and with what results?

San Francisco is a big city, and its characteristics will change but slowly. Nevertheless, recently there have been some important forces in addition to those generally in operation. From within there are such people as the Rev. Dr. Aked, Bishops Hughes and Nichols, Jack London, Charles Tenney Jackson, Miriam Michelson, and a number of others, and the great University of California, which has the largest body of undergraduate students in America and strong graduate departments. Those parts of Greater San Francisco-Oakland having less of the Argonaut characteristics are rapidly gaining, and within a couple of decades will outnumber and constantly more deeply influence the main city. From without, the majority vote which the remainder of the State brings to San Francisco has forced woman suffrage, suppression of race-track gambling, throwing off railway political domination, etc. A red-light abatement act would have become a State law a few months ago had not a referendum petition held it up until the November, 1914, election, and San Francisco has already abolished its notorious Barbary Coast. By the initiative, next fall, California will vote on State-wide prohibition.

Thus Greater San Francisco-Oakland is quite rapidly reducing the advantages which Los Angeles urges so strongly, of a "wholesome atmosphere" for family life, although probably two or three decades will be required to secure any degree of parity in this regard between the two great centers.

It will doubtless be a decade before the Chambers of Commerce and Development Boards in Greater San Francisco-Oakland and Central California will speed up to the present Los Angeles standard. It will be at least as long before the present unsatisfactorily financed beginnings of an electric interurban system develop to the Pacific Electric standard, and longer before the natural attractions of the Bay region are fully utilized. The cost of living will probably always be lower in the Southland, and the beach advantages will never be offset, although the counter attractions of the northern Sierra Nevada Mountains will be made more and more important.

With rapidly increasing Far Eastern trade, the great San Fran-

cisco Bay must become an increasingly important factor. The route to the Orient from Puget Sound is more foggy and less safe, and from San Diego and Los Angeles, longer. San Francisco is the logical commercial center—the New York—of the Pacific Coast.

The Panama Canal will have an important effect, but for quite a few years much less than many seem to think. For a number of years other Coast cities will probably benefit almost as much as, if not more than, the Bay cities, particularly with lower manufacturing, handling, and ship repairing costs, due to lower wages and rents.

Within the last 4 or 5 years enormous areas in the great Central Valley have been thoroughly developed, from a land-and-water-company point of view, for sale in small tracts to home builders. The grand total is much more than 1 000 000 acres, equal or superior to, in every regard—except environment and convenience—and held at from one-half to one-fourth the prices of, lands with similar crop adaptation in Southern California. Unfortunately, the general idea has not been to use the land, but to build irrigation works and "colonize." Conditions have changed in the two or three decades since the Southland was developed. In 1880, 70% of the people in the United States were on the soil, to-day but 51 per cent. Subduing and farming irrigated lands is by no means a poor man's business, and in general those who have enough capital to succeed will not pioneer.

Already the large companies see the "handwriting on the wall" and are arranging to prepare, plant, and cultivate the land—develop it completely, and then sell farms and orchards, rather than raw land and agricultural and horticultural problems to individuals. A noteworthy instance is the Mills Orchard Company, which is planting about 10 000 acres of orchards, mostly lemons, at about the midpoint on the west side of the Sacramento Valley.

The State is putting $18 000 000 into a network of permanent highways, and the good roads movement is in full swing in the Central Valley.

Several careful students of conditions in California believe that the State is likely to have a single-tax law within the next decade, especially as the initiative is available. In this event the effect on the lands in question would be tremendous.

Taking all these things into consideration, it would seem that,

in about a decade, the great Central Valley will reach the point where Southern California was 20 years ago, and will develop thereafter in much the same way as the Southland is doing. Also, on account of the increasing tendency to concentrate in large cities, the effect on Greater San Francisco-Oakland will be much the same as the development of the southern country has been on Los Angeles. This in spite of the great extent of back country and the consequent development of relatively large local centers such as Sacramento.

Manufacturing on the Pacific Coast must also increase, but whether faster or slower than the agricultural and horticultural interests depends on several factors. Northern and Central California, Washington, and Oregon, particularly, can and will have more and cheaper hydro-electric power than any other place in the United States, and the rates and service will be, and already are, completely controlled and regulated by utility commissions. California has enormous quantities of fuel oil, but its cost is now equivalent to coal at about $4 per ton, and there is little probability of material reduction. The climatic conditions of San Francisco conduce to the highest labor efficiency on the Continent. The local market is important and rapidly increasing. The Panama Canal will make cheaply accessible enormously greater territory. San Francisco is the nearest point in the United States to the potentially tremendous consuming countries in the Far East.

The present high wage rates and unsatisfactory local labor situation make it unwise to expect very great increase in the near future. Nevertheless, it seems probable that the wage "differential" in favor of the other Far West cities will gradually be eliminated, especially in view of the general tendency toward State and National arbitration of labor disputes, laborers' compensation laws, etc.

It is impossible, of course, to evaluate numerically these many factors, but such is always the case in projecting population curves. Regarding Greater San Francisco-Oakland, it would appear that:

1.—There is no probability of excessive rates of growth for the next 10 or 15 years.

2.—The increase for 1910-1920 will be considerably greater than that for 1900-1910, during which the great fire occurred.

3.—The increase after about 1935 will be very rapid for several decades.

Probable Growth of Various Parts of Greater San Francisco-Oakland.—Of almost equal importance for the matter at hand is the division of the total growth between the main, east-bay, and suburban centers.

The influences resulting in the various local growths have been set forth, chief of which is available area. San Francisco has a population density of more than 10 000 per gross sq. mile, and probably 15 000 per net sq. mile. In Marin County, the area available for city development within a reasonable time zone is relatively small and scattered, so that the suburban traffic facilities are, and, for some time, will probably continue to be, below the other local standards. Nevertheless, the climatic conditions are peculiarly attractive and the region so picturesque as to assure great suburban development of a high grade, with excellent transportation service. The population down the Peninsula will be much denser, but always essentially suburban to San Francisco alone. At the south end of the Bay is San José, as old as San Francisco, about 45 miles from it and from Oakland. It is now, and doubtless for several decades will be, a local center rather than a suburb.

The trans-bay cities, Oakland, Alameda, Berkeley, etc., which are physically one city, were the result of San Francisco's natural barriers, in spite of a suburban trip so onerous as to begin to alienate them when well grown. Another vital factor is the mainland transcontinental, and the most important of the local railway terminals and shops. Fig. 9 shows the trans-bay traffic. After the disaster in April, 1906, very large numbers of San Franciscans went to suburban points for temporary shelter, and gradually came back home in 1908. Since then this factor has been unimportant.

Although the East Bay cities are growing at rates somewhat like that of Southern California, and very much faster than the main city, they are also developing a greater independence and self-sufficiency. The social and political conditions existing in these communities are essentially different from those in the main city.

A bridge from San Francisco to the mainland has often been suggested, in fact a bill is now before Congress authorizing one. The trans-bay traffic indicates that such a structure, or a "tube"—which would be more expensive—is not economically justified, and probably will not be for a long time.

The curves, Fig. 10, show the writer's conclusions from consideration of a great number of elements, chief of which, however, are those just given. All urban population estimates for a period of more than one-third of a century have a sharply limited value. Revolutionary changes, due to inventions, different ideals and tastes—such as a real "back-to-the-soil" movement would typify—cannot be anticipated.

II.—Probable Water Consumption.

Average Daily Consumption Per Capita.—The per capita consumption of water seems to be constantly increasing in all great cities of the United States, and at a somewhat slower rate in most European cities, largely due to higher standards of living. When meters are installed, large water wastes stop, causing an immediate drop in consumption, but an increase at once begins. There is no reason why liberal use should not be encouraged. This necessity and luxury of modern life costs less than any other, when compared with the real comfort it affords. At any rate, the increased consumption in American cities has greatly exceeded the expectations of 20 years ago.

San Francisco has a relatively low per capita consumption, which, in spite of the cool summers, warm winters, and foggy climate, can hardly continue. Wages are high, and people spend freely for comforts. Outside San Francisco proper there will be a marked increase in the use of water for lawns and gardens.

John R. Freeman,* M. Am. Soc. C. E., concludes that the use in Greater San Francisco 50 years hence will be "125 to 150 gallons per capita per day, although by that time substantially every service is metered." The probable curves for all component parts and for the entire Metropolitan District are given in Fig. 11. If anything, the quantity for San Francisco proper is higher in comparison than for the other parts of the district.

Probable Gross Requirements of Greater San Francisco-Oakland and Its Component Areas.—Ordinarily, combining the population and daily per capita consumption curves gives the gross water requirement, but, in semi-humid and arid territories, this is not quite the case. In this district a considerable area is devoted to agricultural purposes, not a little of which will be used in this way,

* "The Hetch Hetchy Water Supply for San Francisco, 1912;" Report by John R. Freeman, San Francisco, July 15th, 1912, p. 79.

even when the population of the district reaches 5 000 000. Until required for city uses, these lands will be cultivated more and more intensively with consequently greater reliance on artificial and controlled watering.

The underground resources made available by wells and the local surface run-off naturally tributary to the region and which can be conserved at a reasonable cost, would be considerably more than sufficient for the arable land. However, much of such water is used in the main and East Bay cities, and it is impossible in considering the water situation of the district to separate the two kinds

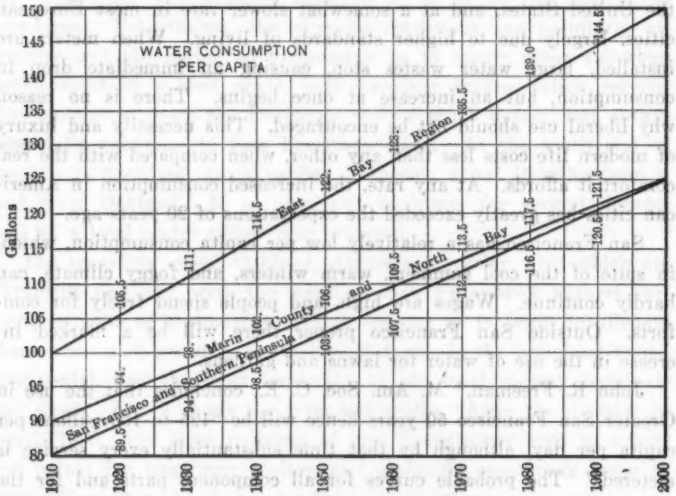

FIG. 11.

of requirements. Consequently, the easy and apparently direct way is ordinarily to obtain the gross water requirement by combining the urban and irrigation uses. Subtracting from these the quantity which may be secured locally, gives the quantity to be brought from a distance.

In doing this, however, it must be borne in mind that well water now available for irrigation and quite satisfactory in quality for domestic purposes will become unsatisfactory for the latter use without effective purification, when the region becomes urban in char-

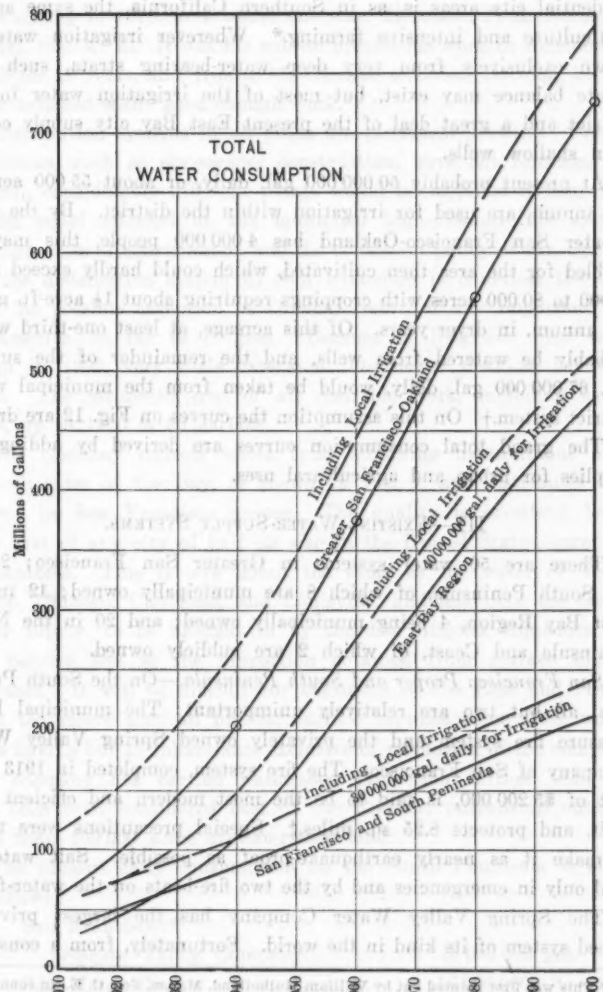
TOTAL
WATER CONSUMPTION
Including Local Irrigation
Greater San Francisco–Oakland
Including Local Irrigation
40,000,000 gal. daily for Irrigation
East Bay Region
Including Local Irrigation
30,000,000 gal. daily for Irrigation
San Francisco and South Peninsula
Millions of Gallons
800
700
600
500
400
300
200
100
0
1910
1920
1930
1940
1950
1960
1970
1980
1990
2000

FIG. 12.

acter. Hence, it is not correct to assume that the duty of water in residential city areas is, as in Southern California, the same as for horticulture and intensive farming.* Wherever irrigation water is drawn exclusively from very deep water-bearing strata, such fortunate balance may exist, but most of the irrigation water in the district and a great deal of the present East Bay city supply comes from shallow wells.

At present probably 50 000 000 gal. daily, or about 55 000 acre-ft. per annum, are used for irrigation within the district. By the time Greater San Francisco-Oakland has 4 000 000 people, this may be doubled for the area then cultivated, which could hardly exceed from 75 000 to 80 000 acres with croppings requiring about 1½ acre-ft. gross, per annum, in dryer years. Of this acreage, at least one-third would probably be watered from wells, and the remainder of the supply, say, 67 000 000 gal. daily, would be taken from the municipal water district system.† On this assumption the curves on Fig. 12 are drawn.

The grand total consumption curves are derived by adding the supplies for urban and agricultural uses.

III.—Existing Water-Supply Systems.

There are 50 water systems in Greater San Francisco; 20 in the South Peninsula, of which 8 are municipally owned; 12 in the East Bay Region, 4 being municipally owned; and 20 in the North Peninsula and Coast, of which 2 are publicly owned.

San Francisco Proper and South Peninsula.—On the South Peninsula, all but two are relatively unimportant: The municipal high-pressure fire system and the privately owned Spring Valley Water Company of San Francisco. The fire system, completed in 1913 at a cost of $5 200 000, is said to be the most modern and efficient ever built, and protects 8.25 sq. miles.‡ Special precautions were taken to make it as nearly earthquake-proof as possible. Salt water is used only in emergencies and by the two fire-boats on the water-front.

The Spring Valley Water Company has the largest privately owned system of its kind in the world. Fortunately, from a construc-

* This was first pointed out by William Mulholland, M. Am. Soc. C. E., in connection with the estimated water requirement of the City of Los Angeles, when the Los Angeles Aqueduct was under consideration. Practically none of the territory marked out for the future development of Los Angeles secured irrigation water from shallow wells.

† C. D. Marx, M. Am. Soc. C. E., adopts 95 000 000 gal. daily for a much greater district, including a considerable part of Santa Clara County.

‡ The City and County of San Francisco has a total land area of 47 sq. miles.

tion point of view, all planning and building for a period of 43 years ending in 1911, were under one man, Hermann F. A. Schussler, M. Am. Soc. C. E. It was his rule to project his work in considerable detail for 10 years, and, true to his German technical training, he was never satisfied with "good enough work."

Distribution systems in all cities are piecemeal growths. Present conditions, such as sky-scraper construction, were not foreseen. In general, considerably different plants would be installed to-day. Due to the relief afforded by the municipal high-pressure fire system, the high class of construction, and the consistent policy of Mr. Schussler, the distribution system in San Francisco is doubtless the best, with the possible exception of Omaha, Nebr., of any important city in America, so far as it goes. Because of litigation over rates, begun in 1903, very little extension into newer districts has been made. The estimated cost of all desirable work, including extensions, up to 1920 is $2 500 000.

The company collects water from adjoining mountainous areas on both sides of the Bay, as shown in Fig. 1, and distributes 95% thereof in San Francisco proper. The quality is excellent, better than that of any city of half its size in the United States, except as to hardness. This is now about the same as the water of Washington, Chicago, Cleveland, and Louisville, and less than the Owens River supply, to be brought in by the Los Angeles Aqueduct. It will become less as the proportion obtained from surface sources increases. It will be difficult for engineers of the East and Middle West to believe that all present surface supplies come from watersheds having but 18 inhabitants exclusive of employees of the company. The company has carried water-shed protection farther than has ever been even considered east of the Rockies, purchasing outright more than 100 000 acres of water-shed land.

The present daily consumption on the South Peninsula is about as follows:

San Francisco:*	
Spring Valley Company	40 000 000 gal.
San Mateo County	16 000 000 "
Part of Santa Clara County	10 000 000 "
	66 000 000 gal.

* In addition wells and Bay supplies are used in manufacturing, to an unknown extent.

East Bay Region.—On the mainland, the important factor is the Peoples Water Company, which supplies practically all the water used in Oakland, Alameda, Berkeley, and several smaller places. Unlike the Spring Valley Water Company, it is the successor of nine lesser companies which entered the field during the past 45 years. These companies, becoming unable to meet the rapidly increasing demand, were successively displaced by stronger ones. According to J. H. Dockweiler, M. Am. Soc. C. E., who has been Consulting Engineer for the Cities of Oakland and Berkeley for a number of years, the history of these attempts to keep pace with the widely scattered and rapidly growing population centers by private capital is a most interesting and curious succession of failures properly to grasp opportunities and protect investments by adequate preparation for the future. According to him, largely as a result of 5 years' relentless and wasteful competition between two such companies, the distribution properties are in a considerable degree superimposed on each other; short-lived materials have been used extensively, and the mains and reservoirs are out of consistent relationship with the topography of the territory and the developed sources of supply. According to his records, 48% of the nearly 900 miles of pipe are 2-in. and less, and only about 30% are of cast iron. The area served is 67 sq. miles, 27 of which are above the 200-ft. contour, and it is divided into sixteen pressure zones or areas, ranging from sea level to 1 000 ft. elevation. The system has 60 000 connections, 95% of which are metered, relatively few fire hydrants, and average receipts of 25 cents per 1 000 gal., with base rates as high as 35 cents.

About 100 blocks of the business section of Oakland are covered by a municipal fire system costing, to date, $136 000. Elsewhere on the mainland the fire protection is, in general, very poor, and in many areas negligible. The situation in this regard is probably as unfortunate as in any important center in America.

About one-third the water comes from the surface supplies of the company, shown in Fig. 1, and almost all the remainder from wells near the Bay shore. According to Mr. Dockweiler, the quality is generally satisfactory, although hardly up to the standard of that supplied to San Francisco. The present daily consumption is:

Peoples Water Company:	
Surface	7 000 000 gal.
Underground	11 000 000 "
Union Water Company	700 000 "
Other systems	3 000 000 "
Private wells, about 4 000	8 000 000 "
	29 700 000 gal.

This is exclusive of irrigation water.

Marin County and North Bay Region.—The four principal systems are the Marin Water and Power Company, the North Coast Water Company, the Vallejo Municipal Water-Works, and the Benicia Water Company, the latter being privately owned. The first two are supplying about 2 000 000 gal. daily from surface sources, and the others furnish water for Vallejo (population, 15 000) and Benicia (population, 3 000).

IV.—Maximum Safe Yields of Existing Sources.

The maximum safe yields obtainable by the complete economical exploitation of the various local developed and undeveloped sources are about as follows:

San Francisco and Peninsula.—The complete development of the Spring Valley Water Company's sources of supply would require creating the Calaveras Reservoir, by building a dam 220 ft. high, work on which is now in progress; two other reservoirs, San Antonio and Arroyo Valle, all in the Alameda Creek water-shed; and the diversion of other Coast streams into the Peninsula reservoir system.

The Calaveras Reservoir, estimated cost, $1 500 000, will have a capacity of 53 000 000 000 gal., or 162 700 acre-ft. The flow-line elevation will be 790 ft., and the contributory water-shed is 100 sq. miles, or with the Upper Alameda water-shed, 140 sq. miles.

The San Antonio Reservoir, estimated cost, about $750 000, will have a capacity of 11 500 000 000 gal., or 35 300 acre-ft. The flow-line elevation will be 455 ft., and the contributory water-shed, 38.7 sq. miles.

The Arroyo Valle Reservoir, partly storage and partly regulating, is estimated to cost about $800 000, and to have a capacity of 13 000 000 000 gal., or 39 900 acre-ft. Its flow-line elevation will be 800 ft.. and the

contributory water-shed, 140 sq. miles. It is estimated that the related development of underground sources in Pleasanton Valley and at Sunol will cost $1 300 000.

The Peninsula development will intercept the run-off from 65 sq. miles of territory, contributory to Pescadero and San Gregorio creeks, and convey it by canals, pumps, and tunnels into the Crystal Springs Reservoir, at an estimated cost of $8 500 000 for 50 000 000 gal. daily, or $170 000 per 1 000 000 gal. daily.

The maximum yield which could thus be economically secured for use in San Francisco proper has lately been the source of much bitter controversy. Since 1910 an extraordinary number of engineers, usually believed to be especially competent in such matters, have made examinations and have reported widely differing results, as indicated in Table 1.

There are complete records of gauge heights and draft on reservoirs as follows:

Crystal Springs	1888 to date.
San Andreas	1870 " "
Pilarcitos (Except totals only for 1889 and 1898-1902)	1867 " "
Lake Merced (With several breaks prior to 1906)	1883 " "

There are run-off and discharge records as follows:

Niles Dam	1888 to 1900.
Sunol Dam	1900 to date.
Sunol Aqueduct at Brightside Weir	1906 " "
Pumping records at Belmont Station (With some breaks prior to 1898)	1888 " "
Calaveras Creek at dam site	1898 to 1908, and 1911 to date.

Fig. 13 shows the continued run-off data at the Crystal Springs, San Andreas, and Pilarcitos Reservoirs, platted on arithmetic-probability scales, according to the method proposed by Allen Hazen, M. Am. Soc. C. E.* The consistency of the data thus shown makes striking the wide variation of engineers' estimates in Table 1.

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 1539.

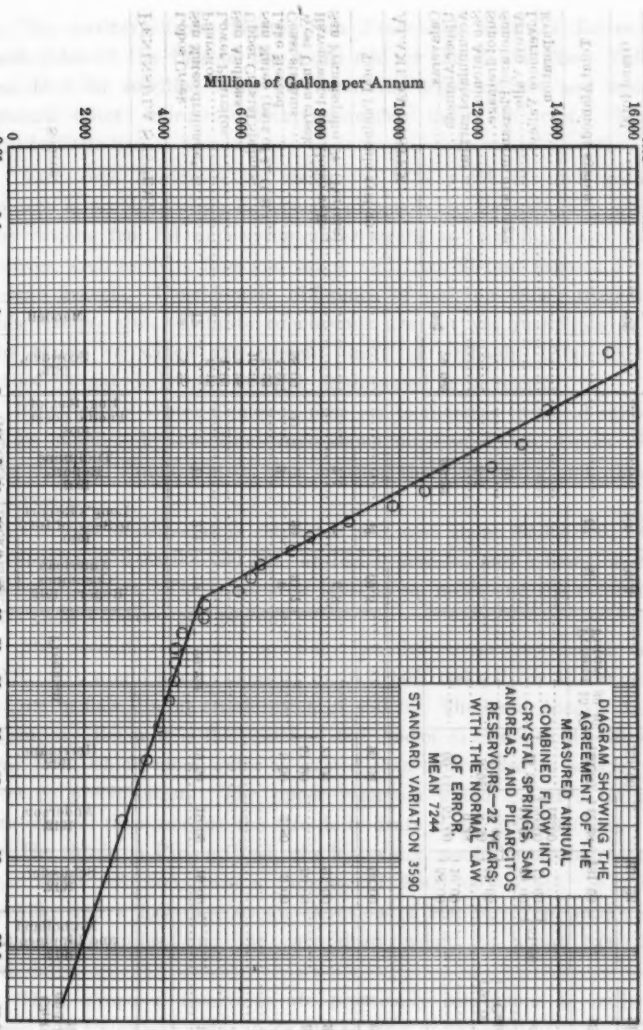
Millions of Gallons per Annum
0
2000
4000
6000
8000
10000
12000
14000
16000
0.01
0.1
1
5
10
20
30
40
50
60
70
80
90
95
99
99.9
99.99
DIAGRAM SHOWING THE AGREEMENT OF THE MEASURED ANNUAL COMBINED FLOW INTO CRYSTAL SPRINGS, SAN ANDREAS, AND PILARCITOS RESERVOIRS—22 YEARS, WITH THE NORMAL LAW OF ERROR.
MEAN 7244
STANDARD VARIATION 3590

FIG. 13.

TABLE 1.—RESOURCES OF THE WATER SUPPLY OF GREATER SAN FRANCISCO-OAKLAND.

Source.	Mendell.	Scowden, 1875.	Grunsky. Eng. Soc. of Pacific Coast, 1905.	Grunsky. Tuolumne Report, 1902.	Dockweiler. Long's Report to Secy. of Int., 1910.	Dockweiler. American Consumers Report, 1912.	Freeman.	Herrmann, 1912.	Anderson, 1912.	Schussler, 1912.	Mulholland and Lippincott, 1912.	Chittenden and Powell, 1912.
PENINSULA SYSTEM.												
Lobos Creek	2	2.25	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
San Mateo drainage*	17	.....	.....	17	21†	19	20–23†	19.50	19.02	19.00	.....	19.50
Pilarcitos	....	5.16	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
Lower Pilarcitos	....	1.93	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
San Andreas	....	4.28	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
Upper Crystal Springs	....	15.00	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
San Mateo (part of L. C. S.)	....	3.23	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
Lake Merced	....	4.267	.....	3	.....	3	.....	3.50	2.80	3.00	.....	.....
Coast streams	....	58.00	20	.....	20	31.7	.....	.....	.....	.....	.....	.....
West Union creek	....	.....	.....	.....	.....	.....	.....	51.20	.....	50.00	.....	50.00
Ravenswood and Aloiso wells	....	.....	.....	.....	.....	.....	.....	21.00	.....	21.00	.....	20.00
San Francisquito Cr. (Portola)	....	.....	.....	7.5	7	5.8	.....	.....	.....	.....	.....	.....
Total Peninsula system	....	.....	.....	.....	48	59.5	.....	95.20	.....	93.00	.....	89.50
ALAMEDA SYSTEM.												
Calaveras	30‡	79.692	.....	25–32	.....	.....	.....	60.14	57.00	58.00	.....	.....
Upper Alameda	....		.....		.....	.....	.....			10.00		
Accumulated surplus	....		.....		.....	.....	.....		9.48			
San Antonio	....	.....	.....	8–5	.....	.....	.....	8.92	8.50	15.66	.....	90.00 includes Calaveras.
Sunol drainage	....	.....	.....		.....	.....	.....	11.36	7.40			
Sunol and Pleasanton drainage	....	.....	.....	12	.....	.....	.....	.....	.....	.....	.....	.....
Arroyo Valle	....	.....	.....	.....	.....	.....	.....	45.38	18.00	30.00	51.50	40.00
Livermore Valley	....	.....	.....	.....	.....	.....	.....		30.00	17.60		
Evaporation	....	.....	.....	.....	.....	.....	.....	.....	12.00	.....	.....	.....
Total Alameda system	....	.....	.....	.....	57	50	Present 15.17 Future 30.40	125.80	142.38	131.26	.....	130.00
Grand total	80§	.....	.....	.....	105	109.5	.....	221.00	.....	224.26	.....	219.50

* Pilarcitos, San Andreas, and Crystal Springs. † Including Lake Merced. ‡ 25 000 000 000 gal. storage. § Includes Coast streams connected with enlarged (Lower) Crystal Springs Reservoir and also surplus from Calaveras, but does not include San Antonio, Arroyo Valle, Pleasanton, or Sunol gravels.

The territory from Greater San Francisco's southern limits on both sides of the Bay to the south end of the Santa Clara Valley can be fully supplied from the remaining local surface and underground water sources within reasonable limits of cost. There will be little excess, however.

East Bay Region.—The maximum safe yield of underground sources is problematical; it depends primarily on the extent to which the various fruitful underlying water strata are supplied from local sources, such as delta fans and cones, and from distant exposed Pleistocene gravels. Considerable difference of opinion exists regarding this matter, also. Definite conclusions are not yet justifiable as to the extent to which the present draft of probably 12 000 000 or 13 000 000 gal. daily may be safely maintained or increased.

The Peoples Water Company proposes to increase the local surface water supplies by building three dams on San Pablo, Upper San Leandro, and Pinole Creeks, thus creating three collecting and storage reservoirs.

The first of these it is estimated will cost about $2 335 000, have a capacity of 6 000 000 000 gal., or 18 420 acre-ft., and a flow-line elevation of 325 ft. The water-shed is 34.65 sq. miles and the estimated safe yield 8 000 000 gal. per day.

The second reservoir, on San Leandro Creek, estimated cost $2 878 000, would have a capacity of 16 700 000 000 gal., or 51 270 acre-ft., and a flow line at an elevation of 470 ft. The water-shed is 32.94 sq. miles. Below it is the present Lake Chabot or San Leandro Reservoir from which more water is wasted than impounded in seasons of average precipitation. The upper reservoir would conserve all the run-off, and thus increase the safe daily draft about 10 000 000 gal.

The third, on Pinole Creek, the last which would be created, would cost about $676 000, have a capacity of 2 300 000 000 gal., or 7 060 acre-ft., and a flow line at an elevation of 290 ft. The water-shed is 10.5 sq. miles, and the safe draft would be about 3 000 000 gal. daily.

These proposed reservoirs are practically contiguous to built-up areas. In time their water-sheds will be built on; at least, it is desirable that the area be available for the rapid growth of population. It would be necessary to filter the water carefully. The utilization of the San Pablo and Pinole sites would affect the annual water replen-

ishment in the delta cones of these creeks where a considerable number of wells are in use. The quantity of water required to be delivered to such regions as recompense, would be about 1 500 000 gal. daily.

The Richmond Water District plans are for a distant supply from the Sacramento River.

Marin County and North Bay Region.—The plans outlined by the Marin Municipal Water District provide for a total safe draft from local sources of 14 000 000 gal. daily, which is regarded as the maximum feasible limit. In the North Bay region the available increase from local water sources is relatively insignificant.

Summary.—The total ultimate safe draft for municipal purposes, of a quality quite satisfactory and well above the present standards for important cities in America, and obtainable at a reasonable cost by complete development of local sources is, then, about as follows:

For San Francisco and Peninsula	180 to 200
" East Bay Region	37 to 40
" Marin County and North Bay Region	14 to 16
	231 to 256

V.—Can Greater San Francisco-Oakland Afford a Sierra Supply?

San Francisco proper has available a disproportionately large part of the local water yield. By complete development, which can be done piecemeal as necessary, it can have an ample supply of excellent quality for at least 75 years to come, exceptionally large local reservoir capacity, and consequently relatively small upkeep and danger of interruption from floods, landslides, or earthquakes.

The city's present assessed valuation is $623 000 000. To acquire the Water Company's properties needed will cost between $35 000 000 and $40 000 000. The city has not finished rebuilding following a $350 000 000 fire. It has a total authorized bond issue of $43 000 000, exclusive of the Hetch Hetchy issue of $45 000 000 which latter is authorized, but not put out. Because of an exceedingly bad transportation situation, it is well started on a programme of taking over and operating all traction lines within its limits. This will require an estimated expenditure of between $100 000 000 and $125 000 000 during the next 25 years. It has no elevated or subway systems, though these are needed and cannot long be postponed. It is a rapidly growing city, with

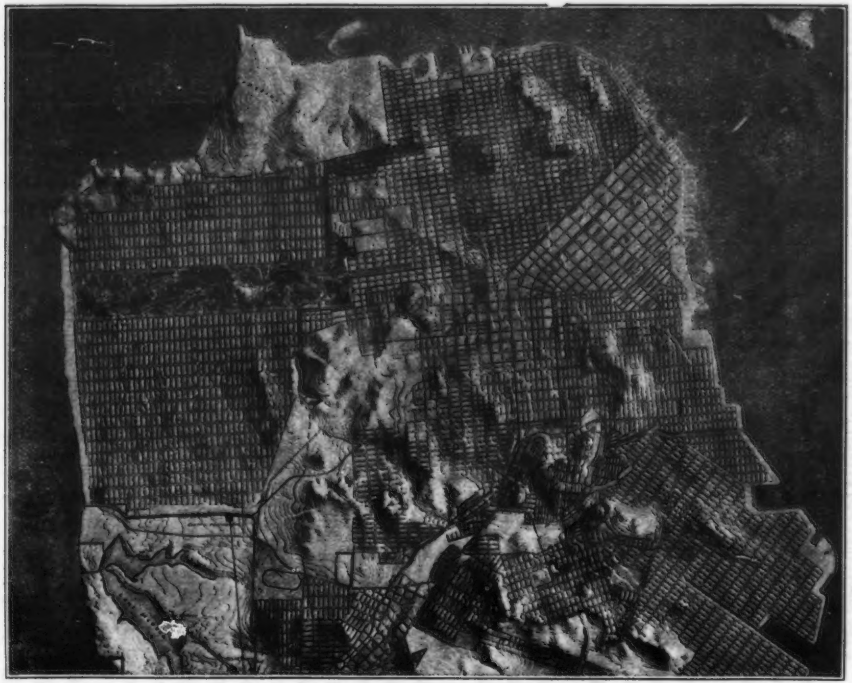

FIG. 14.—RELIEF MAP OF SAN FRANCISCO, SHOWING STREET LINES IN RELATION TO TOPOGRAPHY.

all the financial needs that this implies, yet is already surpassed in population by Los Angeles, which, 15 years ago, was not even admitted to be a rival. Its topography necessitates heavier costs for improvements than in almost any other city in America, and more to be done.* The local labor and political situation makes labor costs high. The temperament of its people makes concerted action difficult and progression slow and along indirect and expensive lines.

The outstanding indebtedness of Los Angeles is about 4% greater than that of San Francisco, with about seven-eighths the assessed valuation. However, Los Angeles owns its local water system, and has finished bringing in the Sierra supply. San Francisco, at least, will have to double its outstanding bond issue, in order to acquire the Spring Valley Water Company's properties, and the proposed completed Hetch Hetchy supply will add more than twice as much. At the best, then, to put the two centers on a proper basis of comparison, including the Sierra supply, San Francisco's bonded indebtedness would be more than four times that of Los Angeles. As to ability to care for such indebtedness in the future, Los Angeles is, in this regard, as in population, growing tremendously faster.

Under such circumstances, the real question is: Can San Francisco afford the luxury of a Sierra water supply at an immediate expense of more than 10% and a total ultimate cost of at least 16% of its present assessed valuation?

San Francisco's natural destiny as the commercial center of the Pacific Coast is beginning to be threatened. At least, until this matter is settled one way or another, San Francisco proper will be the largest and richest portion, and the natural leader of the Bay communities in this struggle.

In competition for newcomers, and in other elements causing a city's growth, is the water supply, as long as it is well above the American standard in quality and is delivered in ample quantities, at reasonable cost, and under satisfactory pressures, an aggressive and

* The topography and natural features of San Francisco are such that it could have been made one of the most beautiful cities in the world. Unfortunately, like many other places, it grew "Topsy fashion," and the layout is rectangular, quite regardless of the configuration of the land. There result grades as high as 22 per cent. The opportunity which the great fire of 1906 presented for re-designing the most important part of the city was not utilized. The City Engineer's office recently stated that $26 000 000 was the cost of certain improvements, necessary under the circumstances, but which would have been entirely avoided had the city's planning been adjusted to the topography, as might have been done, and that this expenditure was but the beginning of a series which from time to time must be made.

obvious factor, as compared with local transportation facilities, modern marine and railway terminals, paving, parks, boulevards, speedways, making areas spoiled by local development available for higher classes of city building, etc.?

Los Angeles, with 86% of the assessed valuation, as a matter of absolute necessity, has brought in a supply of 250 000 000 gal. per day from the Owens River at a cost of a little more than one-fourth of San Francisco's present programme. The quality of this water will, at best, be no better than the local supplies of San Francisco. Perhaps most important of all, however, is the fact that from the very outset, every drop of the water brought in will be used, and at prices well above the existing sinking fund and operation charges per million gallons daily. The reason that this is possible in Los Angeles, and utterly impossible as to Greater San Francisco-Oakland, is that irrigation is absolutely essential in the environs of the southern city, and irrigation water even at the city base rate of 9 cents per 1 000 gal., would be only $29 per acre-ft., which is well within the commercially feasible limit in that territory; on the other hand, in the environs of San Francisco-Oakland, little water is imperatively required, and none could be sold at even $5 per acre-ft. The fact that irrigation requirements for lands adjoining Los Angeles are practically the same as the urban needs will be when the city spreads out over them, has already been mentioned.

If engineering is attaining the best results for the expenditures, this question is an engineering one of the highest type. It is the proper valuation, for a great city's purposes, of two sources of water supply.

East Bay Region.—The situation of the mainland cities is entirely different. The maximum possible development of the local water sources available would supply them for 10, or, at most, 15 years. Their distribution system, according to Mr. Dockweiler, is more than unsatisfactory, and their growth is already seriously hampered by the water situation. It would be advantageous for them, were San Francisco proper to "pull their chestnuts out of the fire." The rivalry between them and San Francisco is such that the latter would doubtless do nothing of the kind were the matter generally thus understood.

These communities face a serious situation. Their combined assessed valuation is only about $250 000 000. The ideal move would be for each section of Greater San Francisco-Oakland to acquire its

respective water system, extend and rebuild its distribution system, as the case may be, pool the local water supplies, and join in bringing in a common distant supply when necessary. Unfortunately, the local political and social situation makes such an arrangement improbable.

It is essential to realize that the water problem of Greater San Francisco-Oakland is very much more one for the mainland cities, than for San Francisco proper.

Marin County and North Bay Region.—What has been said of the East Bay cities is true of Marin County and the North Bay region, except on a much smaller scale.

VI.—Efforts Toward Municipal Ownership.

San Francisco, Oakland, and Denver are the only large cities in the United States which do not own and operate their water-works. According to present California laws, rates, extensions, financing, and service of all public utilities, outside of incorporated municipalities, are regulated by the State Railroad Commission. In November, 1914, the State will vote on the proposal to put all public utilities under the exclusive control of the Railroad Commission, and it is generally believed the change will be made.

Since 1880 municipal councils or supervisors have regulated, practically as to rates only, their public utilities. As elsewhere, this has been unsatisfactory, and at various times more or less serious steps have been taken to acquire or construct local municipal water systems.

San Francisco Proper.—The City had reports made by the late Gen. B. S. Alexander, Corps of Engineers, U. S. A., and the late George Davidson, Hon. M. Am. Soc. C. E., then of the U. S. Geodetic Survey, jointly (December, 1871); the late T. R. Scowden, M. Am. Soc. C. E. (May, 1874); and the late G. H. Mendell, M. Am. Soc. C. E., Col., Corps of Engineers, U. S. A. (1876-77). After the Mendell report the City officials tentatively offered $11 000 000 for the Spring Valley Water-Works' properties on the peninsula and the Company asked $12 500 000. The negotiations were dropped at this point.

On January 8th, 1900, a new charter went into effect, in which was declared the purpose of gradually acquiring the City's public utilities. Under it, the City Engineer at that time, C. E. Grunsky, M. Am. Soc. C. E., prepared plans for an entirely independent system, taking water from the Tuolumne River in the Sierra Nevada Moun-

tains, generally known as the Hetch Hetchy project. He estimated the cost of bringing 60 000 000 gal. per day to the Bay at $31 603 330, and the distribution system at $8 807 000. After 12 years of effort on the part of the City, the necessary rights of way, under severe restrictions, have recently (December, 1913) been granted by Congress in the passage of the so-called Raker Bill. On January 5th, 1914, the City officially accepted the grant and the conditions it carries.

In 1909 the voters passed on an alternative proposition for a bond issue of $45 000 000 to build the Hetch Hetchy project, including a distribution system, or an issue of $65 000 000 to bring in Hetch Hetchy water and buy the Spring Valley Water Company properties for $35 000 000. The latter lacked about 1 100 votes of the necessary two-thirds majority, and the former was carried.

On May 11th, 1908, the Secretary of the Interior at that time, the Hon. James R. Garfield, issued what is known as the "Garfield Permit," granting the rights of way requested by the City in the Yosemite National Park. This required the development of other portions of the project before utilizing the Hetch Hetchy Valley, a wonderful canyon of rugged beauty second only to the Yosemite Valley. On account of many protests, Secretary Ballinger, in March, 1909, acting on the advice of a committee consisting of Dr. George Otis Smith, Director of the U. S. Geological Survey, and L. C. Hill and E. G. Hopson, Members, Am. Soc. C. E., Supervising Engineers of the U. S. Reclamation Service, ordered the City to show cause why the Hetch Hetchy Valley should not be eliminated from the permit. A board of Army Engineers was appointed by President Taft, to assist and advise the Secretary in the matter. The members were Col. John Biddle, Lieut.-Col. Harry Taylor, and Col. Spencer Cosby, Members, Am. Soc. C. E. The appointment of this Board was probably due to President Taft's well-known penchant for the Army. Doubtless, none of these military engineers would urge his qualifications by training or experience as an expert in irrigation, hydroelectric, or municipal water supply matters.

At the request of the City Engineer, Marsden Manson, M. Am. Soc. C. E., Mr. John R. Freeman was called in by San Francisco to assist it in presenting its case. He associated with him eleven other members of the Society and President J. C. Branner of Stanford Uni-

versity. Afterward, M. M. O'Shaughnessy, M. Am. Soc. C. E., succeeded Mr. Manson (September, 1913), and shouldered much of the responsibility. Mr. Freeman fundamentally changed the Hetch Hetchy project as theretofore outlined.

The hearing was held by the Secretary of the Interior, the Hon. W. L. Fisher, on November 25th to 30th, 1912, and it is greatly to be regretted that the proceedings have not yet been published. The Army Board presented a report, largely the work of H. H. Wadsworth, M. Am. Soc. C. E., Assistant Engineer, Rivers and Harbors Branch, Corps of Engineers, U. S. A., on February 19th, 1913. On March 1st, Secretary Fisher decided that he was not authorized to act, and recommended Congressional action. His successor, Secretary F. K. Lane, concurred in this. Hence, there followed the introduction and passage of the Raker Bill.

On December 31st, 1913, the City of San Francisco filed suit to condemn the properties of the Spring Valley Water Company. A year before, tentative negotiations had been broken off, with the Company asking $37 500 000 and the City offering $37 000 000.

East Bay Cities.—On the mainland, Oakland and Richmond are the only communities which have made definite efforts to acquire water systems and supplies.

Oakland.—In 1902 Rudolph Hering, M. Am. Soc. C. E., made a report for a Citizens' Committee on the water supply for Oakland. He advised that 15 000 000 gal. per day of underground water could be obtained from the delta cones of Alameda Creek. In 1903, Desmond FitzGerald, Past-President, Am. Soc. C. E., reported to the same committee, and the next year a Board of Engineers, consisting of A. M. Hunt and J. M. Howells, Members, Am. Soc. C. E., and Mr. F. C. Turner, reported to the Oakland authorities on the adequacy of the Bay Cities Water Company's project for a local surface water supply from near Mt. Hamilton. A bond issue for taking over and building this project was defeated by the voters in December, 1904.

In the same year the rates of the Contra Costa Water Company (since absorbed by the Peoples Water Company), in Oakland, were fixed by the City authorities at a point so low that the Company appealed to the Courts. The result was the same as in San Francisco, namely, expensive litigation still pending and inadequate extensions of the system.

In 1910 litigation was begun between the Peoples Water Company and the City of Berkeley, but it was soon compromised by minor concessions on the part of the Company.

The formation of a municipal water district is now being considered.

The Richmond Municipal Water District.—This district was formed by an election held December 3d, 1912, covering the region bounded on the south by the Alameda-Contra Costa County Line and on the east and north by an irregular line approximating the eastern edge of the San Pablo Creek water-shed. It contains 56 sq. miles, of which 40 sq. miles are below the 200-ft. contour. The population on June 30th, 1913, was estimated at 15 585, of which 13 575 were in the City of Richmond. The present assessed value in the district is $19 400 000.

The present daily water consumption is 3 400 000 gal., equal to 194 gal. daily per capita, of which 123 gal. are used in manufacturing and industrial pursuits. All water is pumped from underground sources, and almost exactly one-half is from private wells, the draft from which has about reached the available limit. The future per capita use will probably increase to 230, and then gradually drop to 150 gal. per day when the population reaches 150 000. Such large per capita consumption is expected because the industrial growth for many years will exceed the residence and business increase. There is no adequate fire protection.

The engineers of the district, Mr. P. A. Haviland, M. Dozier, Jr., M. Am. Soc. C. E., and F. H. Tibbetts, Assoc. M. Am. Soc. C. E., of San Francisco, have reported exhaustively on two water sources available. These are, a reservoir on San Pablo Creek,* and by pumping from the Sacramento River at Toland's Landing, with an emergency intake at Antioch. The safe draft of 12 000 000 gal. daily can be secured from San Pablo Creek at a cost of about $3 500 000, and a like quantity brought from the Sacramento River, including a mechanical filtration plant, for from $4 000 000 to $5 600 000, depending on the conduit route and the materials used. The immediate development of a filtered river supply for 6 000 000 gal. daily, at a cost of from $2 300 000 to $4 100 000, depending on location and construction details desired, was recommended. The District's

* This reservoir site is the one already mentioned as being owned by the Peoples Water Company.

Board has adopted this recommendation and is arranging with the towns *en route*, Martinez, Port Costa, and Pinole, to join in the project, and a bond election to provide funds will at once be held.

The present rates are from 30 to 35 cents per 1 000 gal., and it is estimated that, if the proposed plans are carried out, the cost will be from 20 to 25 cents, reducing in a few years, through large consumption, to 15 cents.

Marin Municipal Water District.—The present per capita water consumption is 85 gal. daily, a very small quantity for the class of urban development served. This is partly due to the actual inability of obtaining water as desired, but, chiefly to the high cost, which is 50 cents per 1 000 gal. and upward. Such high rates are in considerable measure due to the topography of the country. Early in 1912 the Marin Municipal Water District was formed, covering 115 sq. miles, in which the assessed valuation is $13 000 000. In November of that year A. R. Baker, Assoc. M. Am. Soc. C. E., was appointed Engineer, and M. M. O'Shaughnessy and Edwin Duryea, Jr., Members, Am. Soc. C. E., Engineering Advisers. It is understood that steps will immediately be taken to carry out Mr. Baker's recommendations. These are:

1.—Build a dam on Lagunitas Creek about 4 miles below the present reservoir of the Marin Water and Power Company (Lake Lagunitas), on the northern slope of Mt. Tamalpais, and create a storage capacity of 6 400 000 000 gal., or 19 700 acre-ft. In this way a daily supply of 5 000 000 gal. is expected to be obtained. The water will be taken around the shoulder of the mountain in tunnels and pipes, and delivered wholesale to the various localities from a system of main supply pipes. The total estimated cost of this first installation is $2 000 000.

2.—Take over the properties of the Marin Water and Power Company and the North Coast Water Company, and in time develop all their resources, together with the proposed new work, to the full capacity, estimated at 14 000 000 gal. daily, at a total cost of about $5 000 000.

3.—An alternative plan, not recommended, of pumping water from wells alongside the Russian River, about 25 miles from the north end of the district, and conveying it through Santa Rosa, Petaluma, and other communities *en route*, each of which would probably become customers. The cost for installations of 5 000 000 and 15 000 000 gal.

daily, from this source, would be very nearly equal to those respectively recommended.

The local distribution systems, like those of the East Bay cities, are without broad design, and leave very much to be desired, particularly in the matter of fire protection and permanency of materials.

VII.—Essential Factors Affecting a Choice of Distant Water-Supply Sources.

Far-sighted planning for a large additional supply should consider and provide for the water needs, not of the East Bay region, nor of San Francisco proper, but of Greater San Francisco-Oakland as a whole. Because there is a strong sentiment in the East Bay cities against joining with San Francisco in any political or financial enterprise, at least as long as San Francisco has a predominating influence, and because of a rapidly growing realization that in many ways the two sides of the Bay are competitive, it is possible the East Bay cities will act independently.

Quantities Required for Various Dates.—The additional quantities needed by the several portions of and for all Greater San Francisco-Oakland are about as given in Table 2.

TABLE 2.

	1920	1940	1960	1980	2000
San Francisco Proper and Southern Peninsula	10	50	100	155	200
East Bay Region	12	83	215	350	480
Marin County and North Bay Region	2	5	12	23	37
Totals	24	140	327	528	717

It will probably be economically desirable to obtain from outside sources about the quantities given in Table 3.

TABLE 3.

	1920	1940	1960	1980	2000
San Francisco Proper and Southern Peninsula	0	0	0	5	50
East Bay Region	3	46	178	312	443
Marin County and North Bay Region	0	0	1	9	23
Totals	3	46	179	326	516

Doubtless it would be profitable, certainly desirable, and at least a very nice thing, to furnish water to the urban communities along or close to the conduit or aqueduct line, and to the mutual interests of all concerned that excess water be used for irrigation along that line, until needed in Greater San Francisco-Oakland. Under existing laws, this would endanger a servitude being placed on such excess water. It must be remembered, however, that even the State Constitution can very easily be changed—especially with the initiative, referendum, and recall in effect.

Discounted Present Value Only Fair Method of Comparison.—Distant sources have the serious disadvantage that, as single-conduit installations cost much less than double or triple lines of equal capacity, larger and more expensive works than present needs demand are economically required; and interest charges on initial investments mount up startlingly. Those sources which are adaptable to gradual development have a marked advantage. Greater San Francisco-Oakland may obtain water, satisfactory as to quantity and quality, from so many different sources, that, to make fair comparisons, it is necessary to outline the expenditures of all kinds, construction, operation, maintenance, depreciation and renewals, and incidental incomes, at specific dates, and reduce them to present values, using probable interest rates.

Location of Local Storage.—Another disadvantage of distant water sources is increased likelihood of damage to the conveying conduit or aqueduct. As the distance increases, other things being equal, not only is the risk of sudden interruption greater, but also the time required to make repairs, the cost of patroling, and of up-keep. To avert serious consequences in case of interruption, it is necessary to provide large, near-by storage facilities. The greater the risk the more storage is required.

Fortunately, San Francisco proper has Crystal Springs Reservoir on the Peninsula (capacity 19 300 000 000 gal., or 59 200 acre-ft.), and in a few years will have the Calaveras Reservoir (capacity 53 000 000 000 gal., or 162 000 acre-ft.), on the east side of the Bay; the East Bay cities have Lake Chabot (capacity, 4 840 000 000 gal., or 14 800 acre-ft.), with the possibility of creating the larger Upper San Leandro Reservoir (capacity, 16 700 000 000 gal., or 51 270 acre-ft.), at a cost of $3 000 000. Marin District now has Lake Lagunitas and Phœnix Lake (capacity, 382 000 000 gal., or 1 186 acre-ft.), and probably within a few years

another larger storage basin which the Marin Municipal Water District plans to build (ultimate capacity, 6 400 000 000 gal., or 19 700 acre-ft.).

The location and elevation of these local storage facilities have an important bearing on the selection of a distant source.

Earthquake Zones and Faults.—The seismic disturbances which have occurred thus far in California have been *tremblors*, and not very serious matters, provided—but only provided—their likelihood is frankly recognized, the facts concerning their occurrence thoroughly ascertained, and methods to circumvent them carefully and systematically worked out. President J. C. Branner, the eminent geologist of Stanford University, urges scientific, sane, and calm study of the "active faults" in the earth's crust in California. When these are located it will be possible to build so that when future earth slips occur, the damage will be negligible. If the wriggling line of the "1906 Fault" had been accurately known, the Spring Valley Water Company would not have had its pipe line laid over it, the water supply would not have been cut off, and the disastrous fire would not have happened.

Crossings of Waterways, Bay, and Mt. Diablo Range.—There are serious natural obstacles to be overcome in bringing water from distant sources into San Francisco. With the Sierra Nevadas to the east there is the wide San Joaquin Valley, the River, and the Mt. Diablo Range. The first of these prohibits a gravity line, except by using very long inverted siphons under high pressure, the last necessitating pumping against high head over the lowest pass, or long tunneling in very bad ground crossed by "active faults." To bring water from the northern Sierra Nevadas, requires crossing the narrower but equally low Sacramento Valley, the River, and the Carquinez Straits. All Sierra Nevada sources necessitate taking the water for San Francisco proper across San Francisco Bay. With southern sources Marin County can only be supplied through a tunnel more than a mile long under the Golden Gate. The water here is 300 ft. deep, and the crossing would probably be somewhat similar to the one under the Hudson River at Storm King on the Catskill Aqueduct.

Many local engineers fear the difficulties of tunneling the Mt. Diablo Range, regarding it as a formation exceedingly difficult and expensive to pierce. Mr. Freeman, on the other hand, regards the difficulties it presents as much less than those of the Carquinez Straits

crossing, which latter he regards as offering such tremendous obstacles alone as to dismiss immediately from serious consideration all sources which would involve it.

Conservation.—There can be no doubt that supplying municipal needs is the highest use to which water can possibly be applied. That is by no means saying that other useful requirements should not be considered at all until such highest use is fully satisfied. All needs of a region should be considered in relation to all supplies, and adjustment made to secure the maximum total economic benefits to society. This is so obvious that taking the trouble to state it seems waste of time. Nevertheless, it is usually adroitly ignored and avoided for the reason that the greatest total benefits to society may mean individual lesser benefit or greater cost to a powerful interest. The total benefits and costs are rarely divided in like proportions. To this fact is due conservation and the conservation policy—now generally accepted in theory and almost as generally sought to be avoided in individual cases.

In California, as in all the Far West, because of climatic conditions, water is more important than fertile soil and strategic position. The water resources of the Central Valley and the northwestern part of California, sooner or later, will be so conserved and apportioned as to produce the maximum possible benefits. No use can be so high as to interfere permanently with, or prevent, such a result. Unfortunately, no definite steps have yet been taken toward such an end, and this fact alone seems to justify the National conservation movement. It is extremely unsafe, however, for any interest to assume that such indifference will continue. The world moves and, in the Far West, it moves rapidly. Indeed, it is startling to look back only 10 years and see how the viewpoint of the general public has changed with regard to the exploitation of natural resources and how it has already been crystallized into fundamental changes in legislation. Any person, corporation, or municipality is most short-sighted to make investments without weighing carefully the probable changes in public opinion and consequent legislation, designed to produce a higher degree of intrinsic justice—at least for the period necessary to "unload" successfully.

Furthermore, municipalities are but mutual corporations. In Wisconsin, municipally owned public utilities have for years been regu-

lated as to service, rates, and accounting, exactly like privately owned and operated utilities; and Wisconsin's lead in such matters has been closely followed in the Progressive West.

Irrigation.—Next to municipal use comes irrigation. In discussing the growth within California, the agricultural history of the Central Valley has been touched on and the radical changes at present under way noted. The American, and particularly the western, characteristic attitude toward experience elsewhere, resulted in irrigationists having to learn by bitter experience the wisdom of the United States Reclamation Service's standard of satisfactory irrigation water supply. This standard is that all water users have ample water at all times when it can be used profitably. It involves storing flood-waters for use when streams are very low or entirely dry during the latter part of the summer. The California Central Valley has been almost the last part of the United States to find itself in this regard.

The tremendous value of the climate and fertile soil in the great Central Valley is still not appreciated. The crop adaptation is marvelous. It is conservative to say that 20 years hence an acre of properly irrigated bare land anywhere in the Central Valley will be worth at least $500, and citrus land in the foot-hills from $600 to $800. It is equally conservative to say that in the central and southern parts of the San Joaquin Valley, particularly, unirrigated agricultural land will be worth not more than from $20 to a possible maximum of $50 per acre.

Power.—Next in order of use is the development of hydro-electric power. California stands near the head of all States in the quantity of developed water-power, and yet only a fraction of its possibilities has been utilized. The cheapest fuel is oil from local fields, now costing about 75 cents per bbl., wholesale, equivalent to coal at $3 per ton, and the supply is by no means limitless. The tremendous economic importance of the power resources of the State is obvious.

The difference between the ultimate value to society of a natural resource and the commercial value of a development of that resource must be understood. Local and general economic conditions have resulted in smaller hydro-electric concerns gladly disposing of their output in blocks at high load factors to the large companies for about ½ cent per kw-hr. One of the largest power corporations in the State, with hydro-electric plants scattered for miles along the western slope

of the Sierra Nevadas, transmission lines gridironing the central part of the State, and a 60% load factor, is generating more than 40% of its energy in steam plants.

Indeed, there is a general misconception of the commercial value of "water rights." Every decrease in cost and increase in efficiency of steam plants reduces the relative advantages of water power. The steam turbine is a notable example. The "value of water rights" has lowered tremendously in the past few years, because of rate and service regulating commissions. The tendency is to regard a water right, not as an asset, but as an exclusive license for self-constituted trustees to develop a public resource and operate it for the benefit of the public, on a percentage. This change of sentiment has practically wiped out the commercial value of a proposed project controlled by a "water filing." Companies doing a tremendous business and having ample credit can wisely carry out enormous developments and secure thereby very low costs per unit of installed capacity. The rapid development and constantly lowering cost of electric generating and transmitting equipment and the progress of the art in general are other important factors.

For example, one of the recent large projects in the State, now more than half completed, is a development in which water will be dropped through six power-houses and generate 165 000 h.p., with a total cost of $130 per installed kw. or $97 per h.p. Even this will be reduced, because water from the lowest plant will irrigate a large body of land and the State Utility Commission will certainly take irrigation water rentals into account in fixing power rates. Obviously, such developments affect the value of all hydro-electric plants and projects looking to San Francisco as a market, if not throughout the whole State.

This matter has been considered at length because the Army Board evaluated the "incidental power possibilities" of the Freeman Hetch Hetchy project—115 000 h.p.—at $51 100 000. This is more than $445 per h.p. On such a basis, the water rights of the power company's complete development, just mentioned, would have to be $37 425 000. A completed hydro-electric installation feeding into a completed transmission system supplying an actually developed market, is probably worth not more than $125 per h.p.—without such market, particularly, and the means of serving it, much less. Los

Angeles is discovering this in connection with its aqueduct power, as the city itself finds it impossible to use the hydro-electric power at a high average load factor.

Navigation.—The Sacramento River is technically navigable to Red Bluff and the San Joaquin River up to Firebaugh. Both streams are much used in the lower stretches—the former to Sacramento and the latter to Stockton. Beyond these latter points navigability is at present important chiefly on account of holding down local railway rates. Consequently, navigation in this case ranks below irrigation and power uses.

Obviously, any water storage which increases the natural flow at low-water stages improves navigation. Diversion from tributaries and upper reaches of main streams for irrigation is usually, though not always, conflicting. Diversion from higher areas, entirely outside the water-shed, as a supply for Greater San Francisco-Oakland, would always be conflicting, though, with some sources, with very small low-water flow, negligibly so.

In the Sacramento Valley there are several available storage possibilities by which the flow of the Sacramento River at Red Bluff could be increased by 500 sec-ft. for 4 months, and numerous ones to provide such increase below Marysville. In this way, navigation interests could easily and relatively cheaply be protected to permit diverting enough water from any part of the Valley drainage to supply Greater San Francisco-Oakland.

In the San Joaquin Valley the situation is not so fortunate. There the water supply is so deficient, except possibly in the northern end, that the use of storage opportunities in this way cannot be afforded.

Flood Protection.—Last of all—so far as storage is concerned—comes flood protection. The 1 750 000 acres flooded in the Central Valley must always be protected by levee systems almost exclusively. Of course, storage for almost any use would help.

Senator Newlands, the author of the Reclamation Service Act, has recently introduced a bill in Congress providing $60 000 000 per year for 10 consecutive years for flood protection, of which one-twelfth is for the Sacramento and San Joaquin Rivers, and the California Reclamation Board has outlined a $50 000 000 project for the Sacramento Valley. Spending such tremendous sums for reclaiming

FIG. 15.—RELIEF MAP, SHOWING PRECIPITATION OVER CENTRAL CALIFORNIA.

FIG. 16.—RELIEF MAP, SHOWING RUN-OFF OVER CENTRAL CALIFORNIA.

bottom-lands and neglecting to prevent waste of such citrus lands through unwise distribution of water resources is foolish.

All Uses Interwoven.—All these utilizations of the Central Valley's water resources are inter-dependent. To secure the greatest total benefits to society offers no serious problems in statecraft or engineering—only an intelligent general plan. In some sections there will then be water to waste, in others a deficiency. In any event, all comers, no matter of how high or of how low degree, or of what precedence their claims, will sooner or later be made to conform with such a general plan.

California Precipitation and Run-Off.—Figs. 15 and 16 show the probable mean precipitation and run-off curves for the State, based on all the data yet collected.

It is more than unsafe to design hydraulic works in California from mean annual figures, because of dry cycles often 6 and 7 years long. The precipitation, speaking generally, is due to storm centers coming over the Pacific Ocean and passing on to the east. When the center enters the United States near Puget Sound, practically no precipitation results in the southern half of the State, but when the path lies farther south, say, at San Francisco, the entire State receives rain. For some reason these storms often pass well to the north for several successive years. Each year, in addition, there are usually three or four relatively unimportant storm centers which pass over or near San Diego. Nevertheless, it is said that the records, kept by the Franciscan Fathers at the Mission in Santa Barbara for more than 130 years, show 1 year with absolutely no measurable precipitation.

As a consequence, in the southern end of California all significant storms are general, and in dry years so gentle and distributed that the percentage of run-off is almost incredibly small. Going northward, such conditions are less noticeable, but, throughout the entire State, especially as far north as Sacramento, long dry-year cycles frequently occur.

There are few run-off data in California prior to 1901, when the State and the U. S. Geological Survey began co-operating in systematic stream gauging. There are shorter cycles of dryer years before this period, however. This fact must be kept in mind in connection with the run-off curves given. For example, one would make a grievous

mistake by using all the stream gaugings after 1902 in Fig. 17, and assuming them to be typical. There were few reservoir records prior to 1901 in the Sierras, like those at Sweetwater and Cuyamaca.

Irrigation Water Requirements.—In determining the excess, if any, of available water above the needs for irrigation in any territory, there are the two factors, irrigable area and the local duty of water. The first is simply measured, except that in some cases the greatest utilization of a large region's water resources may require modifying the natural limits, such as water-sheds.

The local duty of water, however, is as yet a matter of judgment. It varies with soil types, present and future crops, drainage, character of supply, metered or unmetered service, and extent of cultivation. Very little Middle and Northern California data are enlightening, because very few supplies are really what they should be. Water rentals on a quantity basis are the rare exception. The almost universal disposition of irrigators to use too much water is well known. Few supplies are dependable, particularly during the latter part of the irrigating season, when streams are at the lowest. This results in using excessive quantities during times of plenty, to store water in the soil, a wasteful plan, unsatisfactory in many ways. Thus water-logging and temporarily ruining large areas is the general experience in almost all western irrigation projects. Averages of current practice, therefore, are thoroughly unsafe guides; the extreme limits are too far apart.

The various State Agricultural Experiment Stations and the U. S. Department of Agriculture have made numerous experiments to check such collected data. Unfortunately, these data do not have great practical value, just as the results of acceptance trial tests on steam-power plants are not safe bases for estimating output cost per horse-power per annum.

An important economic factor is the degree of soil cultivation. When the Sweetwater Reservoir was empty and the precipitation very small, citrus trees near San Diego were kept alive with only 3½ in. of irrigation water per annum, but at a cultivation cost normally prohibitive. Dry farming could be practiced successfully over vast areas where it is economically unjustified under conditions now and for a long time obtaining. The two greatest problems in America are to stop the disproportionate growth of cities and to improve the labor situation on the farm. Assuming that 1 acre-ft. of water per

annum and a cultivation cost of $25 per acre will produce as much as 3 acre-ft. of water and a cultivation cost of $5 per acre, is the water saving justifiable? Except where the water supply must be extended

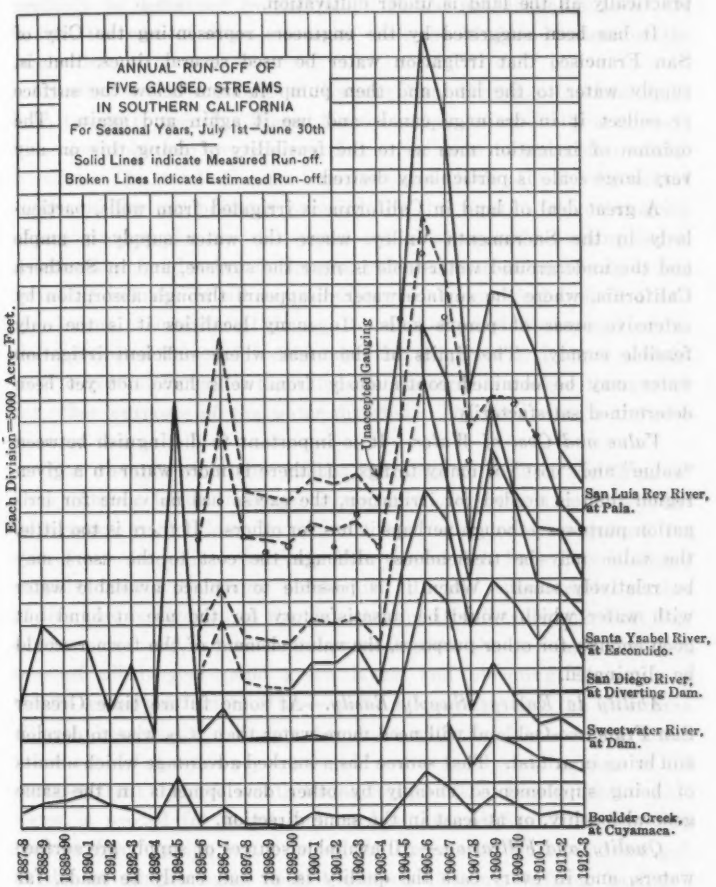

FIG. 17.

over exceptionally valuable land, obviously it is not. Experimental or theoretical and economical minimum water duties are very different things.

Many irrigation engineers believe the only safe guide is the actual water use over large successfully cultivated areas where conditions are essentially similar, where the water supply is metered, and where practically all the land is under cultivation.

It has been suggested by the engineers representing the City of San Francisco that irrigation water be used several times, that is, supply water to the land and then pump it from below the surface or collect it in drainage canals and use it again and again. The opinion of irrigation men as to the feasibility of doing this on any very large scale is particularly desired.

A great deal of land in California is irrigated from wells, particularly in the Sacramento Valley, where the water supply is ample and the underground water-table is near the surface, and in Southern California, where the surface water disappears through absorption by extensive areas of porous soils. In many localities it is the only feasible supply. The limits of the areas where sufficient irrigation water may be obtained continuously from wells have not yet been determined satisfactorily.

Value and Cost of Water.—It is important to distinguish between "value" and "cost" of many things. If there is more water in a given region than is needed for irrigation, the excess has no value for irrigation purposes—though perhaps it has for others. If there is too little, the value may be tremendous, although the cost to the users may be relatively small. When it is possible to replace available water with water which would be unsatisfactory for the use at hand but good enough for other purposes, the value element of the former would be eliminated.

Ability to Enlarge Supply Easily.—At some future time Greater San Francisco-Oakland will need more water than it is wise to develop and bring in at first. That source has a marked advantage which admits of being supplemented cheaply by other developments in the same general locality, or at least in the same direction.

Quality, and Filtration.—All available sources of supply are surface waters, and in every case the quality is, or can easily be made, far above what is generally regarded as satisfactory for large cities. Some are better in this regard, others in that; some will require immediate filtration, with others it would be unnecessary. The disposition is to insist more and more on treatment of all surface supplies, even when long

storage in reservoirs is possible. Quality standards in everything are rising, and filtration to remove coloring matter and minute organisms, even when, for hygienic reasons alone, it would be unnecessary, will probably be demanded by the public in the not distant future. Of course, the cost of filtration varies with the kind and extent of the impurities.

Sentiment.—The people of Greater San Francisco-Oakland have been taught to regard a Sierra Nevada supply as markedly better and more desirable than any other, and this fact must be given due weight.

Cost.—The cost to Greater San Francisco-Oakland is the discounted values of the expenditures required for the rights and construction of any project plus the discounted values of operation, maintenance, deterioration, and renewal charges, plus the cost of antagonistic and resentful feelings of all communities which may feel despoiled and the danger of intentional damage to much-exposed works, and minus the discounted value of incidentals such as power and water rentals.

The cost to society is the value (as the term is used in this paper) for other purposes of the water diverted. The total cost is the sum of these two.

The cost per thousand gallons delivered to him is the thing of chief interest to the average citizen, and of particular and peculiar importance to the East Bay communities. As already explained, the tide of newcomers into the State from east of Salt Lake City is almost entirely toward Southern California. If any considerable portion of this tide is diverted to the Bay regions, it will be by the East Bay communities, and, in the endeavor to do this, the striking attractiveness of all-the-year-round green lawns and blooming flowers is a vitally important factor. These media, all accentuating the California climate, have resulted in Los Angeles largely by reason of the meter water rate of 9 cents per 1 000 gal. and a flat sprinkling rate of 0.9 cent per front foot per month for all lots 150 ft. or less in depth. Less water is needed for lawns and gardens around San Francisco Bay than at Los Angeles, but, even so, the upper cost limit per 1 000 gal. at the consumer's meter cannot exceeed 15 cents without having a serious effect on the area of lawn in the East Bay district.

It is a luxurious thing to have a high mountain water supply, but it is not an obvious point in competing for the Eastern and Middle West newcomers to the State. In this day and generation, to urge a

pure water supply is about as impressive as to dilate on satisfactory police protection, or paved streets—these things are assumed as a matter of course. Green lawns and blooming flowers the year round, and especially in winter, made possible by California's distinctive climate, is constantly impressive to to-day's tourists and to-morrow's fellow-citizens, and these features must be practically universal to compare at all favorably with the southland.

It is a simple matter to take a base rate of 15 cents per 1 000 gal., meter measurement, the present or prospective consumption at any given time, the cost of operation, taxes, etc., and, with any assumed rate of interest, ascertain the capital investment which may be expended on water supplies without jeopardizing this matter of lawn development. Suffice it to say that any proposed water development on a large scale, which would make water cost more than 50% greater than the rates obtaining in Los Angeles, is practically beyond the means of the East Bay cities. This point is of little importance in San Francisco, because there are notably few and small lawns, and even with free water from now on, that city never could be made comparable in that way with Los Angeles.

VIII.—Possible Distant Sources of Supply.

The following additional sources of supply have been suggested: Eel River, Putah Creek, and Clear Lake and Cache Creek, from the northern Coast Range region; Sacramento and San Joaquin Rivers, from the delta region of the Central Valley; and McCloud River, Indian Creek, Feather, Yuba, and American Rivers, Lake Tahoe, Mokelumne, Stanislaus, and Tuolumne Rivers, from the Sierra Nevadas.

A.—Coast Range Sources.

The Coast Range sources are shown on the map, Fig. 18.

Eel River.—The water is not needed locally for irrigation. The length of conduit is much less than that for any of the Sierra Nevada sources, and, as a supply for only the East Bay cities and Marin County, is apparently well worth more careful consideration than it has received. In the construction of the project, the Carquinez Straits crossing should be made sufficiently large to serve as well for the maximum possible additional supplies, such as the McCloud, the Indian Valley, and the Feather River projects, when any or all of these may be needed.

FIG. 18.—RELIEF MAP, SHOWING POSSIBLE COAST RANGE SOURCES OF SUPPLY.

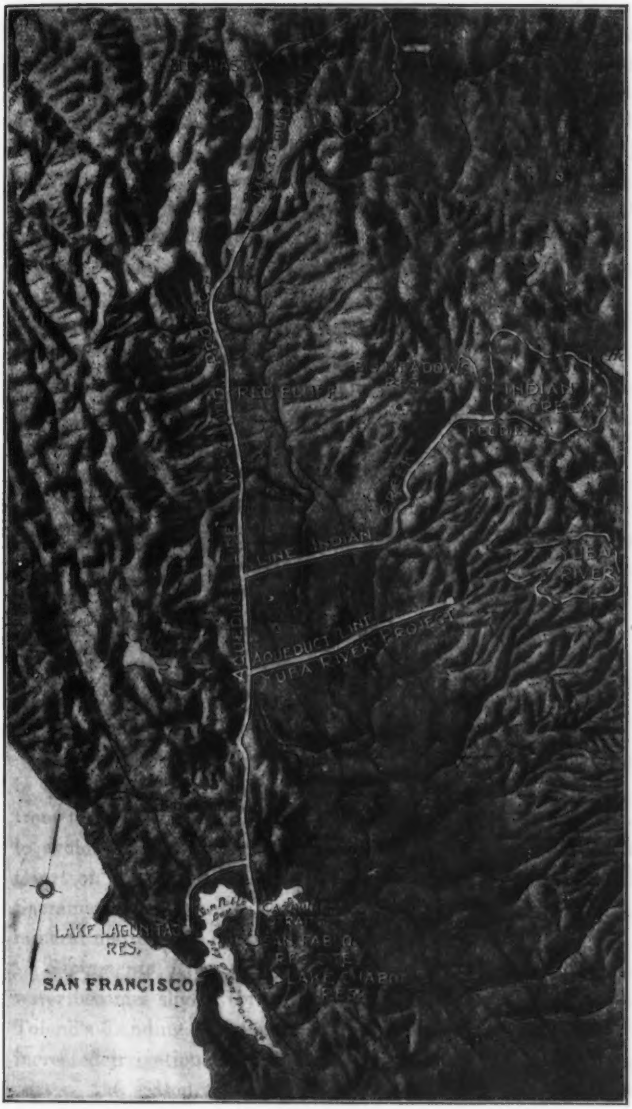

FIG. 19.—RELIEF MAP, SHOWING POSSIBLE NORTHERN SIERRA SOURCES OF SUPPLY.

Above the Gravelly Valley Reservoir site, where 180 000 acre-ft. capacity could be created by a dam 140 ft. high, is a drainage area of 326.5 sq. miles. The safe draft is estimated at 180 000 000 gal. daily, or, with a conduit of 225 000 000 gal. daily capacity, and using the large terminal storage in Greater San Francisco-Oakland, 200-000 000 gal. daily. The conduit would be so near the hydraulic grade line that reinforced concrete pipe could be used for much of the distance. The elevation of the intake is 1 000 ft., and the length of the conduit to San Francisco, *via* Golden Gate, 125 miles. No cost estimates are available.

Putah Creek.—Putah Creek is the closest mountain source. With the excellent Monticello and Guenoc Reservoir possibilities, a safe draft of about 150 000 000 gal. daily could be secured. The conduit could follow closely the hydraulic grade line to Carquinez Straits and be 60 miles long to Lake Chabot, back of Oakland. The water should all be used locally for irrigation.

Cache Creek and Clear Lake.—Within the possible limits of lake-level regulation, 600 000 acre-ft. of storage could be obtained in Clear Lake, with which a safe draft of about 195 000 000 gal. daily would be possible. This water would not be of the best, and it should all be used for irrigating tributary valley and foot-hill land.

B.—Delta Sources.

The Delta sources are shown on the map, Fig. 20.

San Joaquin River.—The low-water flow of the San Joaquin River is so small that an intake would have to be constructed where inflow from the Sacramento River could be assured, and yet be high enough to avoid salt water from the Bay. Mr. Wadsworth suggests Clifton Court on the Old River. The supply would have to be filtered. The Sacramento River water is clearer, of better quality, softer, has less mineral matter, and a much less seasonal variation.

Sacramento River.—At the mouth of the Sacramento River the water becomes slightly brackish late in the seasons of low run-off. At Toland's Landing, 5 miles above, it is always fresh. In the future, increased irrigation use will tend to reduce the flow during the early part of the season, but will probably slightly increase the low-water flow, though storage for power development and other purposes will materially increase the low flow. It is highly improbable that sea

water will ever get materially farther up stream than at present. Mr. Allen Hazen reported in 1912 that the "Sacramento River water can be filtered so as to produce a good potable water at all seasons of the year, removing the turbidity, color, and results of sewage pollution." The resulting hardness would be from 56 to 62, or about two-thirds that of Washington, Cleveland, Chicago, Pittsburgh, Cincinnati, or Louisville, and 80% more than New York or Philadelphia.

The filtration plant and an emergency intake would be near Antioch, as planned for the Richmond Water District. The total length of the line recommended by Mr. Hazen, starting about 8 miles farther up the river, at Rio Vista, running thence directly to filtration works at Antioch, and thence to Walnut Creek, East Oakland, Alameda, and across the Bay to Potrero Point, San Francisco, is about 54 miles.

Estimates on this project were with a $3 wage scale for an 8-hour day and present railway freight charges on material from the East. Taking dates for installations of 133 333 333 gal. daily, except tunnels built originally for 400 000 000 gal. daily capacity, as 1914-18, 1936, and 1975, respectively, interest at 4½%, and assuming that all plants would begin running at capacity immediately on installation, at a cost of $17 per 1 000 000 gal. for operation, maintenance, depreciation, and renewals of plant, the present value of the cost of this source is estimated by Mr. Wadsworth as $51 700 000.

Of course, no one would seriously consider installing works of 133 333 333 gal. daily capacity for a community now fully supplied and using only 70 000 000 gal. daily, nor begin to operate them to full capacity immediately on completion. Caution must be used, therefore, with this figure of $51 700 000.

The water diverted would have no value for any other purpose whatever. No power could be generated, and practically no water sold *en route*.

C.—Sierra Sources.

The Northern Sierra sources are shown on the map, Fig. 19.

McCloud River.—The McCloud River rises on the south side of Mt. Shasta, and, joining with the Pit River, forms the principal tributary of the Upper Sacramento River. It has a remarkably steady flow for California, and a minimum discharge of 1 200 sec-ft., or 770 000 000 gal. daily—amply sufficient for all possible needs. The quality is excel-

FIG. 20.—RELIEF MAP, SHOWING POSSIBLE DELTA SOURCES OF SUPPLY.

FIG. 21.—RELIEF MAP, SHOWING POSSIBLE SOUTHERN SIERRA SOURCES OF SUPPLY.

lent. Much of the water-shed is in the Mt. Shasta National Forest, and practically all the remainder, 400 000 acres, is offered for $3 000 000. All storage necessary from hygienic considerations could be had in local reservoirs. The water is not needed for irrigation, and there are storage possibilities on the river itself and elsewhere in the drainage area of the Sacramento River above Red Bluff which would more than compensate for any purpose for any quantity Greater San Francisco-Oakland could wish to divert during low-water stages. It is doubtful whether the City would ever have to do anything of the kind—certainly not for years. The comparison of this with other Sierra Nevada sources is thus practically one of cost only.

An aqueduct to Lake Chabot, in Oakland, would be 212 miles long. Mr. Wadsworth's estimate for the discounted present value of an initial installation of 260 000 000 gal. daily, 1914-20, and an additional 140 000 000 gal. daily, in 1973, is $52 500 000.

No power would be developed along the aqueduct line, but, on the project and within the controlled tributary water-shed, about 130 000 h.p. can be developed at attractive costs. The conduits, almost entirely of gravity type and of reinforced concrete, would be along the western edge of the Sacramento Valley for the entire length of the latter, and would pass close to many rapidly growing towns and irrigable land having the highest crop and fruit adaptation in the State. All excess water could doubtless be sold along the way from the very start, although not at prices fully proportionate to the cost of the aqueduct and reservoir system.

Indian Creek.—Indian Creek is a branch of the North Fork of the Feather River, with one of the best and most economical reservoir sites in America. A dam 120 ft. high would create a storage of 600 000 acre-ft. and one 220 ft. high and 900 ft. long on top, 2 300 000 acre-ft. (a capacity of 445 000 acre-ft. is equivalent to a yearly flow of 400 000 000 gal. daily). The flow line would have an elevation of 3 680 ft., and the area would be 11 500 acres. The mean annual evaporation is about 3½ ft. The run-off has been gauged from January, 1906, to date, except from January 1st, 1909, to September 10th, 1911. The average annual discharge seems to be about 480 000 acre-ft. The dam site is 4 miles, by good automobile road, from Keddie on the Western Pacific Railroad.

All the water-shed and the reservoir, covering 733 sq. miles, is in the Plumas National Forest, and, except a total of 30 000 acres, which would cost about $1 100 000, is public land. There would be no interference with irrigation or navigation, and fewer legal entanglements than with any other Sierra Nevada or Coast source, except possibly the McCloud. A total development of 140 000 continuous horse-power is possible with the final tail-race at Elevation 1 000. It has recently been investigated by the California Débris and the Conservation Commissions for flood control. It will doubtless soon be used for hydro-electric development. It has no proponents, and no surveys or estimates as a supply for Greater San Francisco-Oakland have ever been made. The conduit line to Lake Chabot would be about 196 miles long.

By State legislation, this opportunity could be held indefinitely available for future use, at no expense except the economic waste resulting from preventing power development below 1 000 ft. elevation. The McCloud Project conduit line would be followed for about half the distance, the Yuba River conduit line would be common for most of the way.

Feather River.—The water-shed of the Feather River is characterized by steadiness (for California) and quantity of run-off, and by great storage possibilities. There is no doubt that it could furnish more water than Greater San Francisco-Oakland will ever need, in addition to all possible irrigation requirements. Unless the intake were at one of the large reservoirs in the head-waters, such as Indian Valley or Big Meadows, the purifying effect of storage would be offset by mixing with run-off from areas almost but not wholly uninhabited. If the supply is to be filtered, the lower Sacramento River, except in point of hardness, is more advantageous.

Yuba River.—There is little natural storage on the Yuba water-shed, except at the highest part of the South Yuba drainage area, therefore the run-off is rapid and irregular. By a system of natural and artificial reservoirs on the South and Middle Forks, a daily supply of 165 000 000 gal. of suitable water could be obtained at reasonable cost. This could be well adapted to a combination with water from some portion of the Feather River Basin, or other sources. The water is always more or less turbid. By making a diversion near the mouth,

400 000 000 gal. daily could be secured, but at so low an elevation that pumping would be necessary and filtration advisable. Less than one-half the quantity could be spared by irrigation interests.

All these Sierra Nevada sources would be brought into Greater San Francisco-Oakland from the north, the conduits crossing Carquinez Straits. This, according to Mr. Wadsworth, would require 5 300 ft. of tunnel and 600 ft. of shafts, and would cost $1 150 000. Piercing the Mt. Diablo Range would be avoided. With the sources following there would be no crossing of Carquinez Straits, but the Mt. Diablo Range would have to be passed.

The Southern Sierra sources are shown on the map, Fig. 21.

American River.—The American River enters the Sacramento River at Sacramento. The Cosumnes, a tributary of the Mokelumne River, lies just to the south, and, owing to the topography, the upper and contiguous portions of the two water-sheds have usually been considered together in outlining power and water projects. A daily supply of about 230 000 000 gal. of excellent water can thus be obtained at reasonable cost, and diverted out of the valley without important injury to dependent irrigation interests. Approximately 62 000 net continuous horse-power could be developed incidentally.

Lake Tahoe.—Lake Tahoe is only mentioned because it seems naturally to suggest itself to many. It can supply daily about 275 000 000 gal. of excellent water, but all of it and much more is needed for the Nevada desert, where, logically and properly, it should be used.

Mokelumne River.—It is estimated that from the Mokelumne River a daily supply of from 130 000 000 to 150 000 000 gal. of satisfactory water could be secured. It is said that a storage of 250 000 acre-ft. is about the limit which can be developed on the water-shed, and that there are 200 000 acres of irrigable land dependent on the stream for water. In such case it is very doubtful whether it would not be against the best interests of society to divert out of the water-shed for any purpose more than half such quantity, or possibly any at all.

Stanislaus River.—Before getting even as far south as the Stanislaus River, the excess of water over satisfactory irrigation requirements has become very small, and here probably passes through zero and becomes minus. By taking into account the water due others by appropriations, it has been computed that about 60 000 000 gal. per day could be diverted out of the water-shed. The present water

laws of California are little less than absurd, and it is most short-sighted to stop with them. Taking into account ultimate irrigation needs, instead of rights, it is improbable that any water is properly available.

Tuolumne River.—The Tuolumne River is the source selected by Mr. Grunsky, City Engineer in 1901, and, because it contemplated transforming the famous Hetch Hetchy Valley into a reservoir, is generally known as the Hetch Hetchy supply. Mr. Grunsky proposed to bring in 60 000 000 gal. per day. In 1912 Mr. John R. Freeman increased the quantity to 400 000 000 gal. per day, or 618 sec-ft., and, among other results, was crystallizing opposition of local irrigation water users and securing by them conditions protecting many of their interests.

The Raker Bill, which Congress recently (December, 1913) passed, granted San Francisco the necessary reservoir, conduit, and other rights of way in, over, and through certain public lands, the Yosemite National Park, and the Stanislaus National Forest. Following the methods of the Conservation Policy, the grant requires:

1.—Filing customary maps, observance of National Park and Forest rules and of prior valid claims, etc.

2.—No sanitary regulations over the water-shed, other than as enumerated (those necessary in any event to protect campers in the Park from having polluted water to use).

3.—Recognition by the City of the prior right of the Modesto and Turlock Irrigation Districts (acreage not to exceed 300 000) to 2 350 sec-ft. of the natural daily flow at La Grange, whenever said districts can beneficially use it, and to 4 000 sec-ft. between April 15th and June 14th.

4.—The City to sell "such amounts of stored water as may be needed for the beneficial use of the said irrigation districts" at cost, with maximum and minimum quantities thereof which can be demanded during any calendar year. These quantities are to be fixed and occasionally revised by the Secretary of the Interior.

5.—The City not to divert out of San Joaquin Valley water for other than domestic and other municipal purposes.

6.—It defines "natural daily flow" as the quantity which "on any given day would flow in the Tuolumne River or its tributaries if said grantee had no storage or diversion works on the said Tuolumne water-shed."

7.—The immediate building of Hetch Hetchy Dam, 200 ft. high, and with a base "capable of supporting said dam when built to its greatest economical and safe height."

8.—The City to sell at cost electric power to said irrigation districts and municipalities within them.

9.—The installation by the City of 10 000 h.p. within 3 years of completing "any portion of the works adapted to the generation of electrical energy;" 20 000 h.p. within 10 years; 30 000 h.p. within 15 years; 60 000 h.p. within 20 years; and all additional possible power as soon thereafter as ordered by the Secretary of the Interior.

10.—The City to construct and forever maintain scenic roads and trails (estimated first cost $1 000 000).

11.—The City, after 5 years, to pay the Government $15 000 annually for 10 years, and thereafter $30 000 annually.

12.—The City to pay all costs of investigations or decisions required under the Act to be made by the Department of the Interior.

13.—The City to convey all its lands in the Yosemite National Park and the Stanislaus National Forest to the United States.

14.—The City to sell water at cost to the United States War Department in or near San Francisco.

The City of San Francisco formally accepted this grant with all these conditions on January 5th, 1914.

Under the Raker Bill, it is impossible to estimate closely either how much water San Francisco may divert or the cost of doing so. It is to finance the complete conservation of the water from the entire water-shed above La Grange and receive thereby the excess over local irrigation needs. The City assumes all the uncertainties. These are:

1.—Quantity of water needed for beneficial use by the 300 000 acres of land during critical years. Opinions vary from 600 000 to 1 050 000 acre-ft. per annum.

2.—Quantity of stored water needed for the beneficial use of the Irrigation Districts.

3.—Quantities the Secretary of the Interior will consider proper for the annual maximum and minimum quantities of stored water the City must furnish. This will be determined by the results of experience in the matter, and by public sentiment at that time, as reflected by the Secretary of the Interior, regarding the propriety of the City

having insisted on going for its water supply to a region where there is a deficiency instead of an excess.

4.—The available storage capacity in the water-shed. This has been usually taken at 800 000 acre-ft. The U. S. Geological Survey adopts 1 000 000 acre-ft. Mr. O'Shaughnessy, the present City Engineer, estimates it as 1 800 000 acre-ft.

5.—Quantity of run-off at the various reservoir sites and diversion points during critical years.

6.—Quantity of storage which the power projects and the irrigation districts will provide without cost to the City.

7.—Construction, depreciation, and operation cost of an undetermined quantity of storage.

The amount which, if invested at 4½% compound interest, in 1914, would construct the Freeman Hetch Hetchy project is estimated by Mr. Wadsworth as $38 901 000. This contemplates the initial installation of 160 000 000 gal. per day, 1914-20 (mean date 1917); 80 000 000 gal. per day additional, 1945-47; and 160 000 000 gal. per day additional (total 400 000 000 gal. per day), 1963-69.

These figures must be used cautiously, however, because:

1.—The City has revised the estimated cost of the first 160 000 000 gal. per day installation from Mr. Freeman's figure of $37 501 400 to about $64 000 000, and probably the figures on the other installations as well.

2.—No renewals of relatively short-lived constructions have been taken into account. There are about 100 miles of steel pipe, some of it under heads as high as 600 to 900 ft.

3.—The Freeman Hetch Hetchy project and the Raker Bill Hetch Hetchy project are very different things.

Net horse-power to the amount of 115 000 can be developed from the ultimate 400 000 000 gal. per day supply, at a cost for power-house and machinery installation of $6 000 000. A large part of this power must be disposed of at cost. For the remainder, the Railroad Commission will fix the rates, at least, practically.

So much for the cost to the City. There is also a cost to society, as any water diverted out of the shed will, within 25 years at most, be needed locally for irrigation (400 000 000 gal. per day (445 000

acre-ft. per annum) is probably enough to irrigate 200 000 acres of citrus lands or 130 000 acres of valley land). Assume the cost of putting such water on the land at $125 per acre, the value of land with water at $500, and without water at $50. The value of water, therefore, would be $325 per acre, 25 years hence. Taking 175 000 acres, the total is $56 875 000, and the present value of this (interest at 4½%) is $18 924 000.

This must also be used with caution, because:

1.—It is doubtful whether the City can obtain 400 000 000 gal. per day from this source to divert.

2.—It is not certain that all the water to be diverted is really needed for local lands. The exact limits of the irrigable area east of the 300 000 acres of the Turlock and Modesto Irrigation Districts have not been definitely determined. Neither is it known how much, if any, of this can be otherwise watered, from other streams, or by pumped wells. Where this may be practicable, the cost to society is the increase in irrigation costs capitalized.

3.—There are no reliable data to determine how many acres the diverted water would irrigate, nor the cost per acre of applying it.

4.—The time in the future, when the lands would otherwise be intensively used, may not be the 25 years taken. It is certain that relatively little of the land would be properly watered immediately, because the irrigationists have not now the financial ability. It would undoubtedly be done in less than 25 years, however.

Relative to this, it is well to remember that ultimately water will be taken from the Sacramento River near its mouth and carried south to help irrigate the insufficiently watered San Joaquin Valley. When this is done, the water will doubtless be taken along the western edge of the valley, where the rainfall and run-off is only a fraction of that on the west slopes of the Sierra Nevadas. It is certain that for at least 50 miles south of the Tuolumne, as far as Kings River, no Sacramento River water will be brought across the San Joaquin Valley to supplement the Sierra Nevada streams in their respective tributary valley areas.

In any event, the present value of the cost to society of the City's selecting this supply is tremendous. This fact, more or less clearly understood, is the cause of much very bitter feeling in the Turlock and Modesto Districts. This bitterness accentuates the un-

fortunate, but in some measure justifiable, feeling common in the City's back country that San Francisco is self-centered and selfish.

Approximate Estimates.—Although there are these several elements of uncertainty, some estimates of the quantity of water which can be secured and of the cost are necessary for comparative purposes, if for nothing else. Assume that 1 500 000 acre-ft. of storage can be provided in about 40 reservoirs very well distributed over the water-shed, and that such a storage system can be operated with 80% over-all efficiency,* including evaporation during the frequent 2 to 2½-year storage periods. From the hydrograph of the Tuolumne River at La Grange, Fig. 22, it would appear that, for the 2½ years ending November 1st, 1913, the total safe draft would have been about 1 200 000 acre-ft. per annum, with all reservoirs empty at the end of the period. During the 29 months the Tuolumne River would be dry below La Grange. Another critical period would have occurred in 1897-99. Judging from the precipitation record beginning in 1868 at La Grange—which is not a satisfactory indicator of conditions, but about the only one available—similar critical periods occurred, ending, respectively, November 1st, 1878 and 1888. In other words, such cycles seem to occur about every 10 years, covering 23% of the time.†

The average cost of 1 500 000 acre-ft. storage capacity would probably be at least $30 per acre-ft.,‡ or a total of $45 000 000. Other interests might be induced to provide half of this, or incidental power might be secured which would cause an equivalent result. Consequently, the cost to San Francisco of such a quantity of storage may be taken at between $22 000 000 and $23 000 000. As the water requirements of the irrigation districts are so much larger than those of the City, and as the ultimate use will develop so much more quickly, the mean date of construction would probably be about 1930. This would make the present value of storage cost $11 620 000.

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* To secure such an efficiency it would be necessary to sacrifice most of the great power possibilities of the water-shed, because of different water requirements. For this reason, the City in 1912 asked the U. S. Forestry Service to permit no water storage in the Tuolumne drainage area above La Grange which the City would not control.

† See also computations by the Director of the U. S. Geological Survey. Senate Doc. No. 246, 63d Cong., 1st Sess.

‡ More probably the average cost for such a quantity of storage would be more than $40 per acre-ft. capacity.

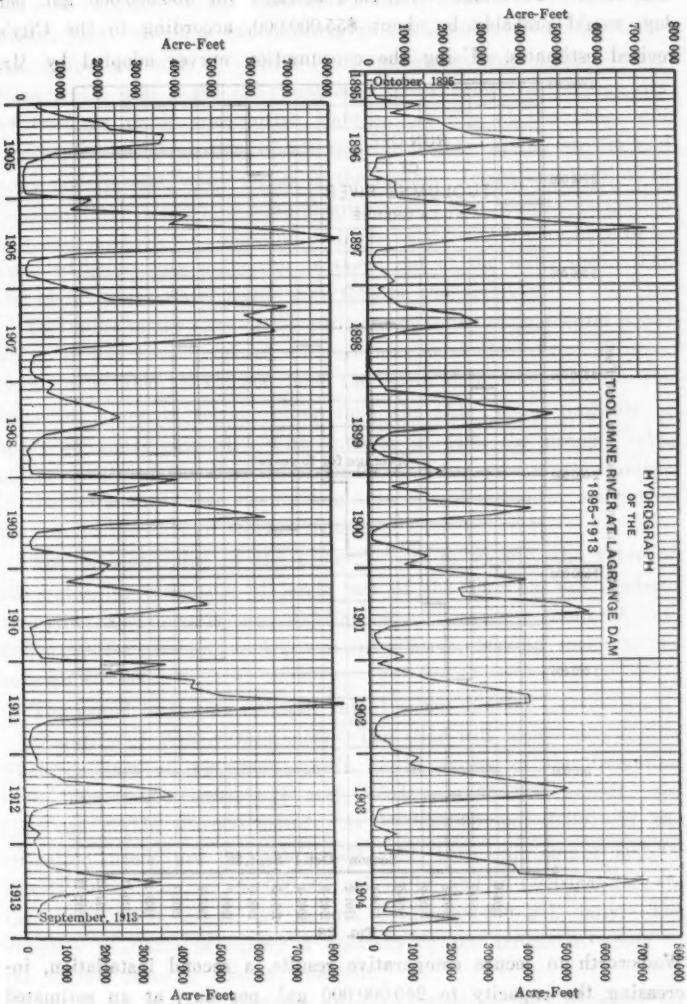
HYDROGRAPH
OF THE
TUOLUMNE RIVER AT LAGRANGE DAM
1895-1913
October, 1895
September, 1913
Acre-Feet
0
100 000
200 000
300 000
400 000
500 000
600 000
700 000
800 000
1895
1896
1897
1898
1899
1900
1901
1902
1903
1904
1905
1906
1907
1908
1909
1910
1911
1912
1913

Fig. 22.

The construction cost of the initial installation, 1914-20, exclusive of the Tuolumne water-shed storage, for 160 000 000 gal. per day, would probably be about $55 000 000, according to the City's revised estimates. Using the consumption curves adopted by Mr.

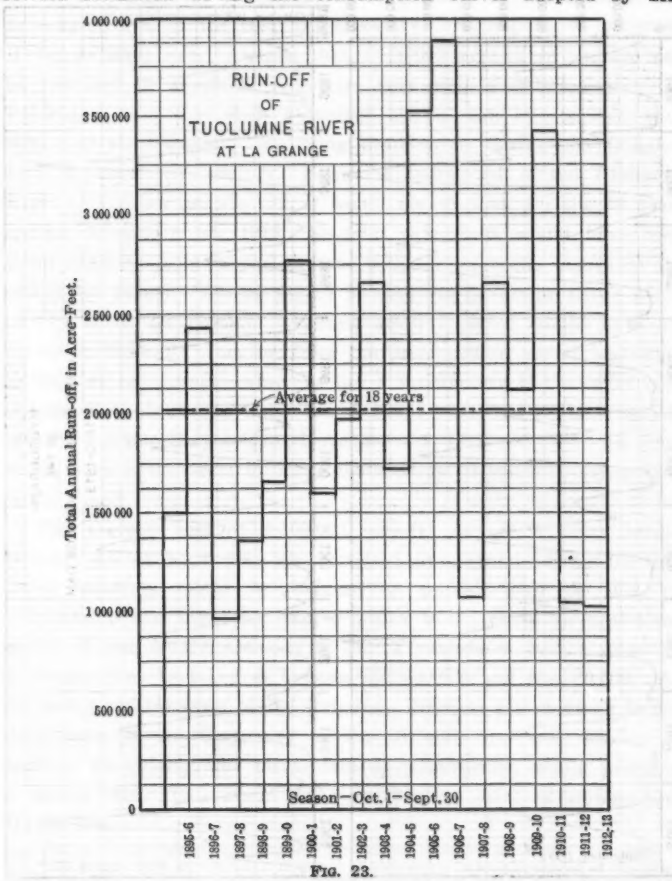

FIG. 23.

Wadsworth to secure comparative results, a second installation, increasing the capacity to 240 000 000 gal. per day, at an estimated cost of $11 000 000, exclusive of the Tuolumne storage, would be required at the mean date, 1946. A second installation, to increase

the capacity to 400 000 000 gal. per day, at a mean date of 1966, will cost, exclusive of the Tuolumne storage, about $13 000 000. The present value of these expenditures is $51 650 000. Nothing is included for renewals of short-lived materials, only initial costs. Plans which will make the Tuolumne River below La Grange dry for from 24 to 30 consecutive months, one-fifth of the time, will necessitate the acquisition of all riparian rights on the stream and many underground water rights in the region. Irrigation districts in California are classed as municipalities, and the Wright Irrigation Law, under which the irrigation districts were formed, may have to be amended before any water can be taken out of the water-shed. The cost of necessary water rights may thus be considerable.

The value to the City of the ultimate quantity of incidental power which it has been proposed to develop can, under the conditions, be no more than $3 000 000, and the mean date about 1925. The value of power to society other than the City would probably be about an equal amount. With interest at 4½% the present value of the credit to the City is $1 848 000. Other power opportunities in the water-shed have been taken into consideration in estimating the cost to the City of storage required.

The present value of the project's cost to society, the value of the water for irrigation of lands east of the Turlock and Modesto Irrigation Districts, is, say, $17 500 000. There is a credit of, say, $3 000 000 for the proposed power installation, leaving society's net cost $15 652 000.

The grand total cost (present value) is $65 454 000.

Quantity of Water Obtainable.—The City will secure per annum whatever part of 1 200 000 acre-ft. is in excess of the irrigation needs of 300 000 acres. If such needs are 750 000 acre-ft., the City can obtain 450 000 acre-ft., or 400 000 000 gal. daily. If the irrigation needs are 900 000 acre-ft., the City can obtain 300 000 acre-ft. per annum, or 273 000 000 gal. daily, and if 1 050 000 acre-ft., then 150 000 000 acre-ft. per annum, or 136 000 000 gal. daily. The quantity available is chiefly a question of the irrigation needs of the 300 000 acres of land in question.

The writer believes that the ultimate irrigation duty of water for this land will nearly equal that in Imperial Valley, California. The two regions have much the same climate, both are essentially alfalfa

and stock districts, with almost identical crop adaptations, and will doubtless have similar social conditions. The rainfall in the Turlock and Modesto Irrigation Districts averages about 9 in., and in the Imperial Valley only 3 in., but in each case practically none occurs during the irrigation season. As an offset, the San Joaquin Valley lands are much more porous than those of Imperial Valley, which are the closest ever successfully irrigated on a large scale. All irrigation water in Imperial Valley is sold on a metered basis, and is used with care and economy. Indeed, practically no drainage system has ever yet been seriously suggested, though the entire area is extraordinarily level, and the soil contained originally so much alkali that the United States Bureau of Soils condemned the region for settlement. The irrigation canals are sealed with a deposition of fine Colorado River silt, so that the seepage loss of the distribution system is quite as low as could be expected from concrete-lined canals.* Imperial Water Co. No. 1 has 101 670 acres under actual cultivation in a compact body, with the crops shown in Table 4.

TABLE 4.

Crop.	Number of acres.	Percentage.
Alfalfa	54 280	54.0
Alfalfa and barley	13 600	13.4
Barley	9 710	9.6
Corn	6 790	6.7
Cotton	6 150	6.0
Corn after barley	3 500	3.4
Melons	3 000	3.0
Miscellaneous	2 880	...
Alfalfa and corn	880	....
Vineyards	820	....

The net quantity of water actually delivered to this acreage—measurements at the highest corner of each 160-acre tract of land—during 1912 was 318 000 acre-ft., and in 1913, it was 325 000 acre-ft.

Safety.—As to relative safety, the presence of "active faults" in the Mt. Diablo Range, which will be crossed by the pressure tunnels, is a matter which many local engineers regard as very serious. Mr. Freeman holds the opposite opinion, and greatly prefers this construction to the Carquinez Straits crossing.

* "Irrigation and River Control in the Colorado River Delta," *Transactions*, Am. Soc. C. E., Vol. LXXVI, pp. 1271 and 1428-1431.

Quality of Water.—Water obtained from this source would be very soft (hardness about 20). All the contributory water-shed lies within the Yosemite National Park, and, therefore, will have no permanent population. On the other hand, with the improved transportation facilities to be made on the terms of the Raker Bill, the number of tourists, summer visitors, and campers on the water-shed will soon become a matter of importance. Taking this into consideration, together with the constantly rising standards in the quality of water supplies, Mr. Allen Hazen and Dr. Rupert Blue, Chief Surgeon, U. S. Navy Department, at the hearing before Secretary Fisher, in November, 1912, agreed in the opinion that complete filtration would be required in about 50 years, possibly sooner. Except for being softer, it would, under existing conditions, be almost exactly on a par with the water now delivered from Crystal Springs Reservoir. Judging from small reservoirs in the mountain regions, there is a possibility that greater trouble would be caused by minute organisms, such as algæ.

Possibility of Enlarging Supply.—If all the Bay communities pool their water supplies, distant supplies will probably not be required before 1945, and by 1990—taking the upper limit of 400 000 000 gal. per day as the quantity which may be obtained from this source—the combined sources will become inadequate within a period of 45 years from their introduction. Long before the latter date, it will be out of the question to secure any more supplies from that general region, and additional water will be obtainable only from the Sacramento River or some of its tributaries, that is, some of the northern sources, the construction of any one of which would be entirely independent of the Tuolumne works.

D.—Combination of Sources.

Obviously, numerous combinations of the various Southern Sierra sources can be made, as well as of those of the Northern Sierras. Some of these may offer considerable advantages.

Conclusions.

Summarizing the salient features of the needlessly complexed political, sociological, and engineering problem, it appears to the writer that the citizens of San Francisco should take stock, and determine:

A.—Whether the water problem of San Francisco is primarily one for San Francisco alone or for Greater San Francisco-Oakland.

B.—Whether San Francisco should attempt to shoulder the financing and responsibility of an entire metropolitan district supply.

C.—Whether the present water system and sources supplying San Francisco proper, when owned actually by the City and properly improved and extended, are not satisfactory and adequate for the needs of the main City for the next two or three generations.

D.—Whether the other portions of Greater San Francisco-Oakland, and particularly the cities of Oakland, Berkeley, and Alameda, are not far more concerned than San Francisco in distant water supplies.

E.—Whether, on sentiment, San Francisco can afford to invest, in a new and distant water plant, sums so vast that many other requirements more vital to the interests of the City must be sacrificed or skimped.

F.—Whether the Hetch Hetchy scheme in its final form is a sound and practical one, from the engineering standpoint, and whether engineers of practical western experience, who are best qualified to pass on its peculiarly western problems, generally endorse or generally question it.

G.—Whether the Raker Act does not strip from the Hetch Hetchy project most of its advantages and assets, leaving the City in the position of "holding the bag" for others.

H.—Whether, all things considered, the only wise line of policy for San Francisco to follow is not:

First, to consummate the acquisition of the present water system, with all its appurtenances;

Thereafter, to take the water situation out of politics by having water matters handled by a Water Board of very wide powers, and entirely independent of all other City authorities;

Thereafter, to improve, extend, and amplify that system to meet fully all the City's needs for 70 or 80 years to come;

And, thereafter, to weigh carefully the relative advantages of entering into the vast Hetch Hetchy scheme or taking up some of the more important sociological problems, with a view to the more

rapid and substantial development of the City itself, and let the other Bay cities work out their own water destiny for themselves.

Also, that the citizens of the East Bay cities should consider and decide, in addition to some of these same things, the following:

A.—Whether they should not acquire the existing water-works on the mainland, and at once improve, extend, and amplify them fully.

B.—Whether, as these cities are really in competition with Los Angeles rather than San Francisco, distant sources necessitating rates exceeding 15 cents per 1 000 gal. can be afforded.

ACKNOWLEDGMENTS.

The writer is indebted to many engineers and others for valuable suggestions and other assistance in preparing this paper. It is desired to extend grateful acknowledgment and appreciation to all of them. Especial thanks are due J. H. Dockweiler and C. E. Grunsky, Members, Am. Soc. C. E., particularly for the use of private reports and data.

DISCUSSION

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F. T. ROBSON,* ASSOC. M. AM. SOC. C. E. (by letter).—The author has mentioned briefly the water supply—as developed and as proposed—of the East Bay shore. Several statements are startling, to say the least, and although the facts as given may be appreciated by engineers who have made a special study of the matter, yet to the Profession at large, and particularly to the local public, the acute water situation of the cities of Alameda and Contra Costa Counties comes as a severe shock. The public has long known of the unsatisfactory financial condition of these water companies, but has not been so well informed as to the water supply.

WATER SUPPLY OF THE EAST SHORE REGION OF SAN FRANCISCO BAY.

San Francisco has dominated everything in Central California for such a long time that the possibility that she will soon be passed in population and industrial growth by the cities on the East Shore has not come to be realized very generally. The cities of the East Bay region have been content to let their sister city make the fight for water alone, expecting to share in the spoils when the fight was won. It appears, however, that it will be too late to wait until San Francisco has brought in mountain water; by that time the growth of the East Shore will have been much retarded. The seriousness of such a situation deserves more emphasis and detailed analysis than that given by Mr. Cory in his paper, and it is felt that some additional facts may prove of interest.

In general, the present supply is as follows:

THE PEOPLES WATER COMPANY.

Historical.—As stated by Mr. Cory, the Peoples Water Company now supplying the greater part of the water used in the mainland cities, is the successor of nine lesser companies which have entered the field in the past forty-five years. Each supplied water to one or more of the transbay cities when they were much smaller than they are at present.

The consolidated plant does not constitute a well-designed and well-built water system. Although materials of an inferior grade were used extensively before consolidation, a big improvement has since been made. Of necessity, a number of features are obsolescent because of the rapid increase in population.

A very large part of the population lies below the 150-ft. contour, but the most desirable residence districts are on the foot-hill slopes to and beyond a 600-ft. elevation. Obviously, the water supply should be divided into at least three vertical zones with storage for each. In general, this has not been done, and could not be expected in a

* San Francisco, Cal.

plant resulting from the consolidation of a number of disjointed systems, largely unrelated, and often duplicated through competition. Mr. Robson.

The main developments were as follows:

The Contra Costa Water Company was formed in 1866 to supply the small City of Oakland, and took water from Temescal Creek, later building the Temescal Reservoir.

Somewhat later the Sausal Water Company began supplying Brooklyn, a part of the present City of Oakland, with water from Sausal Creek.

The California Water Company began operations a little later to supply the Town of San Leandro, from San Leandro Creek, and began the construction of the San Leandro Dam. Both these companies were absorbed by the Contra Costa Water Company, which, in 1875, completed the San Leandro Reservoir (Lake Chabot).

The Alameda Artesian Water Company was formed to supply Alameda, then a small detached town. A group of pumped Artesian wells near Fitchburg was the source of supply. When the population reached 14 000 this became inadequate, and as the only additional supply then possible, because of cost, was the surplus water of the reorganized Contra Costa Water Company, the latter, in 1900, took over the Alameda Company.

The Piedmont Spring Water and Power Company—1892—furnished what is now Piedmont and part of Oakland with water from tunnels driven in the hills back of Piedmont, in Diamond Canyon, and another part of Oakland from wells in Oakland.

The Oakland Water Company, about 1893, was to supply a part of Oakland and the Town of Alvarado, by a pumping plant drawing on Artesian wells near Alvarado. Within a year it acquired the Piedmont Spring Water and Power Company.

The East Shore Water Company was organized about 1898, by interests friendly to the Contra Costa Water Company, and soon after took over the Alameda Water Company. It supplied the higher parts of Berkeley and East Berkeley—then several miles north of Oakland—with water from Wildcat, Cordonices, and Wilson Creeks, from Summit and Berryman Tunnels, and from Pfeiffer Springs. When these sources proved inadequate, the only available increase within the limits of available capital was a surplus of the Contra Costa Water Company, and, therefore, the latter company soon took over the East Shore Water Company.

Prior to the advent of the Piedmont Spring Water and Power Company, in 1892, the Contra Costa Water Company, by acquiring the Sausal and California Water Companies, was in exclusive possession of the field. In November, 1894, the Oakland Water Company (which had acquired the Piedmont Spring and Water Company) first supplied Oakland with water. For the next five years competition

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was severe and wasteful—the Oakland Water Company paralleling more than 100 miles of the Contra Costa Water Company's mains in Oakland, the only territory invaded. In 1899 these competing companies consolidated, the Contra Costa Water Company continuing the business.

Thus, by the end of 1899, the Contra Costa Water Company had, willingly and unwillingly, acquired and merged seven water companies.

The Pinole Water Company was formed in 1905 to take water from the Pinole Creek water-shed and supply a part of the Contra Costa Water Company's territory, the surplus of the latter Company's supply having been exhausted by the greatly increased demand. In a few months, it was purchased and merged—before any works were completed.

In 1906 the Syndicate Water Company was formed by the financial interests behind the Realty Syndicate, which, in 1904, started acquiring and developing on an immense scale the wonderfully attractive foot-hill suburban property on the East Bay side. One part of its programme resulted in the building of the Key Route Ferry and suburban electric railway system, now carrying more than 14 000 000 passengers annually. The deficient and unsatisfactory water supply of Oakland and Berkeley threatened the success of the Syndicate's plans. Rights and holdings were acquired on San Pablo and San Leandro Creeks, and construction was begun to develop enough water to afford—with that of the Contra Costa Water Company—a supply for several years. After some negotiations, the new interests took over the old company and merged under the name of the Peoples Water Company on January 1st, 1907. At that time there were 28 000 connections, mostly unmetered, and 18 000 000 gal. per day were supplied. At present, this corporation has about 60 400 taps, 99% of which are metered; yet the consumption has been kept at about the same quantity as it was 7 years ago. Formerly, a wholesale rate was made to industrial plants and large consumers, but, when the consumption had almost reached the limit of the supply, wholesale rates were discontinued, with the result that many private wells were sunk where there was the slightest chance of securing water. The high price of water has had a bad effect on industrial growth.

Essential Engineering Features.—Fifteen sources of water supply, comprising four surface-water and eleven ground-water sources, have been used during the past 10 years by the Peoples Water Company and its predecessors, as shown on the map, Fig. 24. For the use of this map, and also the profile, Fig. 25, containing data regarding these sources of supply, the writer is indebted to the courtesy of J. H. Dockweiler, M. Am. Soc. C. E.

Lake Chabot, also called San Leandro Lake or San Leandro Reservoir, with a water-shed of 41 sq. miles, was created by an earthen

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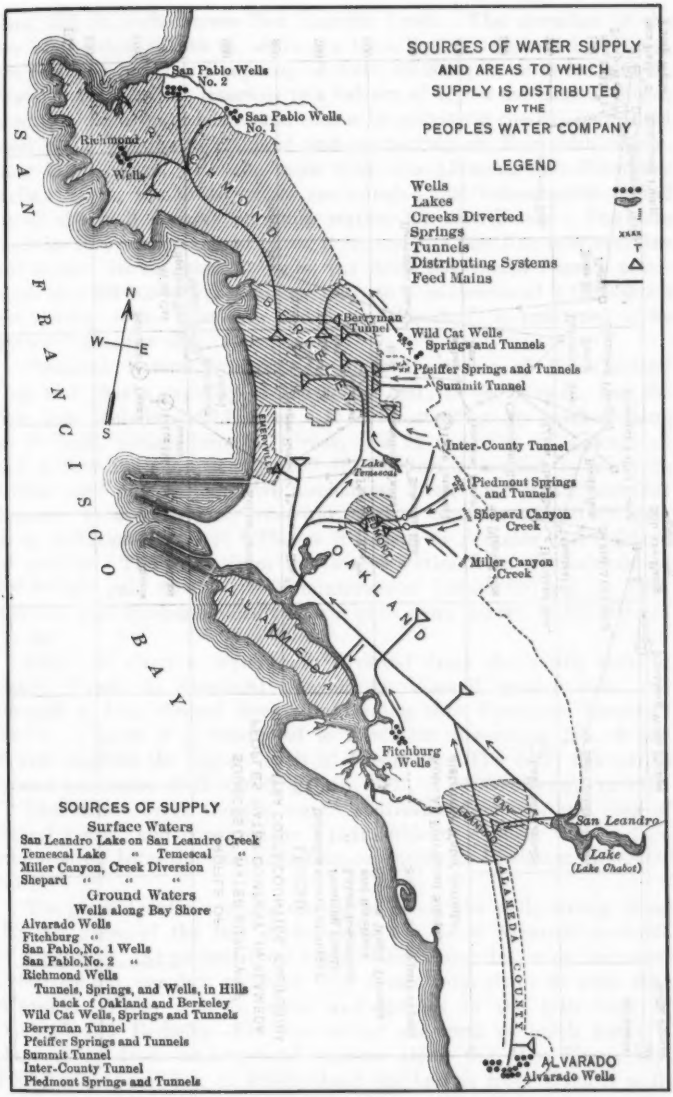

FIG. 24.

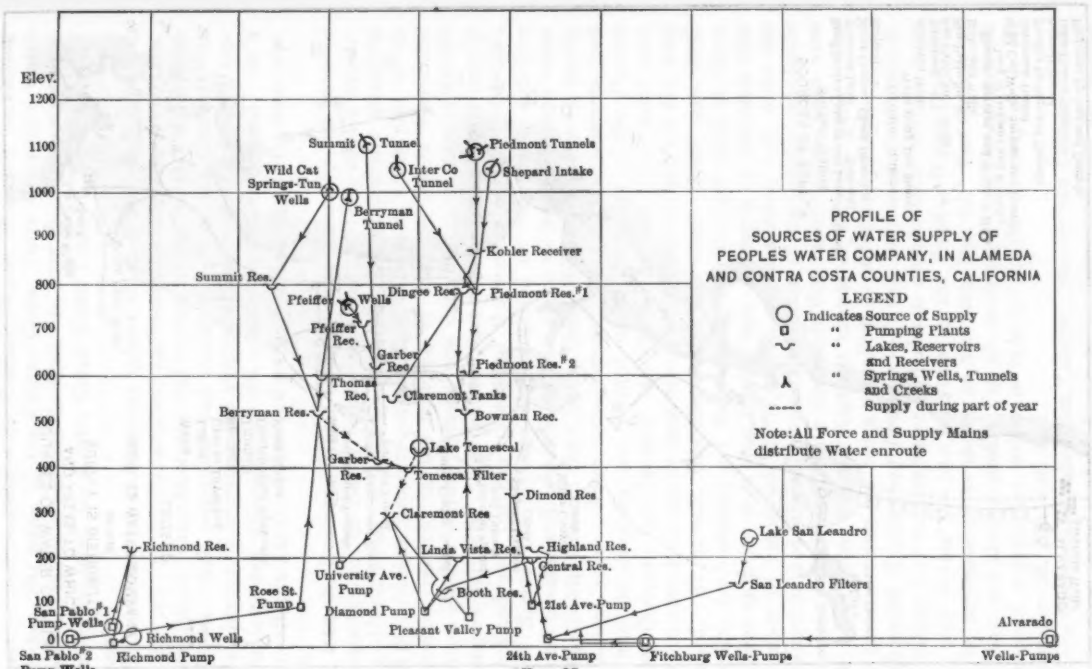

FIG. 25.

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dam, 125 ft. high, across San Leandro Creek. The elevation of the top of the dam is 242 ft. above sea level, and, as reduced by silting, the reservoir now has a capacity of 4 840 000 000 gal., or 14 858 acre-ft. Water flows from the reservoir to a battery of Hyatt mechanical filters, then to a clean-water basin, and thence by gravity to the 38½-in. riveted steel main leading to Oakland and connecting on East 14th Street, near High Street, with the main from the Alvarado and Fitchburg wells. There, the three waters are mingled and subsequently picked up at the 24th Avenue pumping station for distribution. The dam built in 1875 was raised 10 ft. in 1892, and the flow line now contains 447 acres. In the past 10 years, the draft has varied from a maximum of 3 698 000 000 gal. during 1907, to a minimum of 2 152 000 000 gal. during 1911. The safe and dependable draft is presumed to be 6 000 000 gal. per day.

Temescal Reservoir, about 4½ miles northwest of the Oakland City Hall, has a capacity of 150 000 000 gal., or 460 acre-ft., and the flow line includes 16.75 acres. It is created by an earthen dam, 40 ft. high, across Temescal Creek, has 2¾ sq. miles of water-shed, and a flow-line elevation of 436 ft. Water is carried by a 16-in. riveted steel pipe through five mechanical filters (one Hyatt and four Loomis) to a clean-water reservoir, with a capacity of 750 000 gal., at an elevation of 375 ft. Thence it is sent to Berkeley and Oakland by gravity. The draft from this source varies from a maximum of 206 500 000 gal., in 1906, to a minimum of 126 000 000 gal. in 1904. The safe and dependable draft is said to be from 250 000 to 300 000 gal. per day.

Shephard Canyon waters are diverted from the north fork of Sausal Creek, in Shephard Canyon, by a small wooden dam, and through a 7-in. riveted steel pipe leading into Piedmont Reservoir No. 1. Thence it is conducted to Piedmont Reservoirs Nos. 2 and 3, and supplies the higher levels of Piedmont. The draft has varied from a maximum of 32 000 000 gal., in 1907, to 25 000 000 gal., in 1910.

The Miller Creek Canyon water is diverted from the east fork of Sausal Creek, and conveyed by a 2-in. pipe to connect with a 7-in. pipe leading from the Shephard Canyon source to Piedmont Reservoir No. 1.

The ground-water supplies consist of about 150 wells strung along the east shore of the Bay in five localities, from Alvarado to north of Richmond. At present these supplies are being drawn on for about 10 000 000 gal. per day, of which 80% comes from about 35 wells near Alvarado. From tunnels, wells, and springs in the hills back of Oakland and Berkeley—five sets—about one-tenth as much water is obtained as from underground sources along the Bay Shore, say, 1 000 000 gal. per day, of which about one-half is from Wildcat wells and tunnels. These ground-water supplies are not filtered.

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In addition to the four impounding reservoirs, Lake Chabot, Temescal, and Piedmont Nos. 1 and 2 (which have a combined capacity of 5 011 000 000 gal., or 15 384 acre-ft.), there are thirteen distributing reservoirs with capacities of from 158 000 000 to 28 000 gal. (total 240 000 000 gal.), and at elevations of from 80 to 950 ft.

There are five primary and six "booster" pumping stations, with capacities of 16 500 000 and 35 600 000 gal. per day, respectively, with 10 000 000 and 20 000 000 gal. as a daily average. Of the total pumpage about 48% is pumped once, 33% twice, 18% three times, and 1½% four times.

As has been said, the distribution system has about 800 miles of pipe, of which about 48% is 2-in. and less, and 50% 4-in. and less; 35% is cast iron, from 3 to 16-in.; 10% iron and steel, riveted, from 4 to 37-in.; 50% standard screw, from ¼-in. to 8-in.; and the remainder, converse, cement-lined casing and tubing.

Union Water Company of California.

The Union Water Company is the next in importance serving those communities directly across the Bay from San Francisco. It resulted from the activities of the Bay Cities Water Company formed in 1902 to supply water to all the Bay cities.

Historical.—When, in 1901, C. E. Grunsky, M. Am. Soc. C. E., then City Engineer of San Francisco, first reported on a Sierra water supply of 60 000 000 gal. per day, many engineers believed such a quantity of water could be obtained more cheaply from near-by sources. This fact and the friction between the Spring Valley Water Company and San Francisco on the peninsula and the Contra Costa Water Company and the East Bay cities on the mainland, were the underlying reasons for the formation of the Bay Cities Water Company, on October 21st, 1902. The Company acquired rights and large land holdings and outlined a project consisting of two parts—the Mt. Hamilton Project and the Coyote Project. The former is said to be capable of being developed to supply 22 000 000 gal. per day and the latter 40 000 000 gal. per day, or, combined, 62 000 000 gal. per day, approximately the quantity Mr. Grunsky planned to bring in by the initial Tuolumne, better known as the Hetch Hetchy, supply.

The Bay Cities Water Company made offers to both San Francisco and Oakland, but both were declined. Suits were instituted by the Spring Valley Water Company attacking the right to divert water from the Mt. Hamilton Project which is higher up but on the same Alameda Creek drainage area as that Company's Calaveras Reservoir and Sunol filter galleries, and by the land and riparian rights owners in the Santa Clara Valley, on the Coyote Project.

The Coyote Project, as proposed, consisted in collecting and storing surface waters in reservoirs to be constructed, one on Coyote Creek,

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another on Uvas Creek, and a third on Llagas Creek, all draining into the Santa Clara Valley. The decision of the Court in these suits will doubtless make it impossible to use this water in Alameda County.

The proposed Mt. Hamilton development would supply only 22 000 000 gal. per day, and therefore, even though the conflict with the interests of the Spring Valley Water Company did not exist, it would not be attractive, even for the East Bay communities alone. As a matter of fact, the water in question can be conserved more cheaply and delivered to any part of Greater San Francisco by the Calaveras Reservoir of the Spring Valley Company, because the latter is much nearer the mouth of Alameda Creek and the Bay cities.

In the course of these activities, the possibilities of the local underground waters were critically examined. H. L. Haehl, Assoc. M. Am. Soc. C. E., working in conjunction with C. E. Gilman, Assoc. M. Am. Soc. C. E., Manager, Edwin Duryea, Jr., M. Am. Soc. C. E., Chief Engineer of the Bay Cities and the Union Water Companies, and John W. Alvord, M. Am. Soc. C. E., Consulting Engineer, made exhaustive geological surveys, and reached practically the same conclusions, namely:

(1) That there are two distinct and disconnected bodies of underground water within the San Francisco Bay region, one reached by shallow wells less than 200 ft. deep, chiefly supplied by waters absorbed from stream overflow and the rainfall on the local alluvial fans, and to a much less, though considerable, extent by percolations through the joints and fissures of the rock rim and basin of the valley; and the other, locally known as the "Artesian" water, lying much deeper and in large measure supplied by the uptilted gravels of the Pliocene age, exposed at high elevations in the hills on the western side of the valley, where the rainfall is a maximum.

(2) That at the southern end the deep waters are truly Artesian, but the strength of the flow gradually diminishes toward the north, until in the region immediately south of Oakland the deep-seated strata no longer force their waters to the surface.

(3) That analyses of the waters of the two types taken from adjoining wells in general show no similarity, and the seasonal fluctuations of the respective water-tables are essentially different in time.*

(4) That local well-borers call these two underground water bodies "surface" waters and "Artesian" waters, respectively, and cast off

* An interesting account of the geology and geological history of the region may be found in the Report of the Committee on the Geology of the Earthquake, in the Report on the Effects of the San Francisco Earthquake, *Transactions*, Am. Soc. C. E., Vol. LIX, 1907, pp. 216 *et seq.*, and in the Santa Cruz Folio, Geological Atlas of the United States Geological Survey, Washington. Reference may also be made to the discussion on the paper "Rain and Run-off near San Francisco, California," by C. E. Grunsky, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXI, 1908, pp. 522 *et seq.*

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the former in the region between San José and the southern extremity of the Bay, because they must be pumped.

(5) That the water supplied by percolations through the fault zones, joints, and fissures, in the distorted sedimentations of the rock rim and basin of the valley, although not the most important, is a considerable and material source of supply, and cannot be neglected in any proper consideration of the sub-surface water supply.

(6) That draft on the deep-seated "Artesian" waters west of the Alameda or Niles and San Lorenzo fans or "cones", is less than on the "surface" water, and that the former are not dependent for their supply on the surface flow of Alameda and San Lorenzo Creeks.

The Bay Cities Water Company is yet inoperative, and doubtless will never do any business in Greater San Francisco. Instead, the Union Water Company, formed on December 21st, 1910, has put down many wells, scattered over the East Bay region from Richmond to Newark, and constructed several small distributing systems in various parts of the region.

Essential Engineering Features.—The collection works of the Company, shown on Fig. 26, consist of 13 small pumping plants, of which 12 are now in use. In general, each well is an individual pumping plant with a gasoline engine or electrically-driven pump. The Company states that it has a total of 122 wells, in 14 groups, with a 24-hour test capacity of 26 000 000 gal. per day, and a safe daily draft of 19 000 000 gal. Of these wells, 12 are on the San Pablo cone, 65 on the San Leandro cone, 5 on the San Lorenzo cone, 14 on the Niles cone, and the remainder outside these cones. Owing to contracts for watering lands north of Newark, the output from Plants 11 to 14, inclusive (about 11 000 000 gal. per day), are practically not available for use elsewhere. Of the remaining 8 000 000 gal. per day (which it is said may be used in supplying present urban communities) 2 666 666 gal. per day come from 23 wells, Plant 5, and 3 333 333 gal. per day from 33 wells, Plant 6.

The distribution works consist of about 91.5 miles of pipe, forming six different systems, as follows:

Three small systems in Richmond with a total of 10.4 miles in connection with Plants 1, 2, and 3; 2.9 miles connected with Plant 4; 52.6 miles as a network connected with Plants 5, 6, 8, and 9; and 25.1 miles connected with Plant 10; the last two systems are tied together. Almost all the mileage is supplying the south end of Oakland and San Leandro. The pumps deliver directly into the mains, the pressure being controlled by automatic electric devices.

Of the pipe, 39% is 2-in. and less, 72% 4-in. and less, 95.6% 8 in. and less, and the remainder is 10 and 12-in. Cast-iron pipe, from 4 to 10-in., comprises 24%, and the remainder is converse lock-joint and steel. There are 3 100 taps, of which 92% are metered, and 850 000

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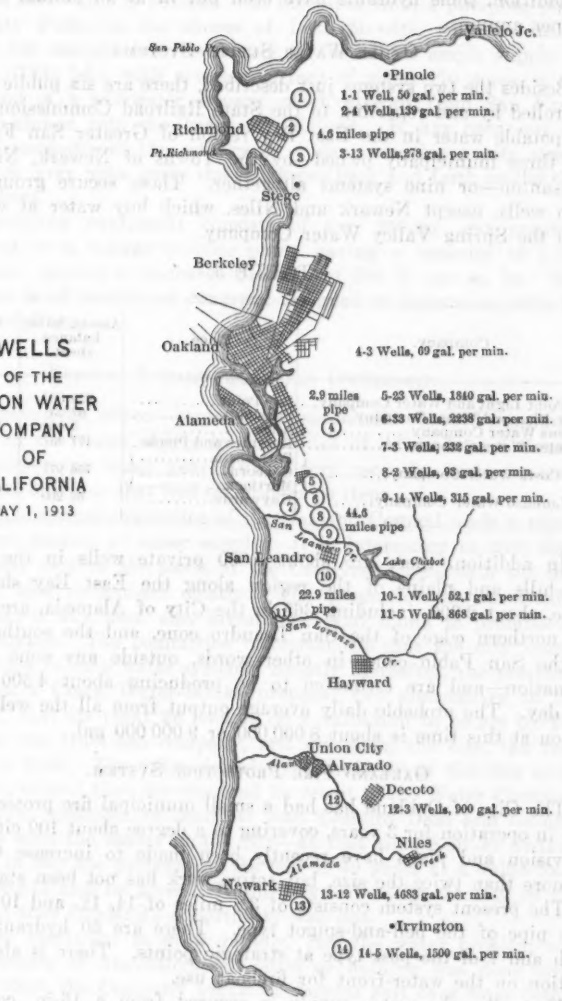

FIG. 26.

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gal. per day are being supplied at a base rate of 32 cents per 1 000 gal. In addition, some hydrants have been put in at an annual rental of $35 per annum.

Other Water Supply Systems.

Besides the two systems just described, there are six public utilities controlled by and reporting to the State Railroad Commission supplying potable water in the East Bay region of Greater San Francisco, and three municipally owned—by the Towns of Newark, Niles, and Pleasanton—or nine systems altogether. These secure ground-water from wells, except Newark and Niles, which buy water at wholesale from the Spring Valley Water Company.

TABLE 5.

Company.	Place.	Assets, latest balance sheet.	Base rate per 1 000 gal.
Bay Point Light and Water Company	Bay Point	$25 000	0.30
Black Diamond Water Company	Pittsburg	67 931	0.20
Citizens Water Company	Niles	5 000	Flat rate.
Hercules Water Company	Hercules and Pinole	177 457	0.50
Port Costa Water Company	Port Costa, Concord, Martinez	756 971	0.30
San Lorenzo Water Company	Haywards	52 200	0.40

In addition, there are about 5 000 private wells in the western foot-hills and plains of the region along the East Bay shore. Of these, about 2 300, including 362 in the City of Alameda, are between the northern edge of the San Leandro cone, and the southern edge of the San Pablo cone—in other words, outside any cone or delta formation—and are estimated to be producing about 4 500 000 gal. per day. The probable daily average output from all the wells in the region at this time is about 8 000 000 or 9 000 000 gal.

Oakland Fire Protection System.

The City of Oakland has had a small municipal fire protection system in operation for 3 years, covering to a degree about 100 city blocks. Provision and plans have recently been made to increase this area to more than twice the size, but active work has not been started.

The present system consists of 3.8 miles of 14, 12, and 10-in. cast-iron pipe of the bell-and-spigot type. There are 50 hydrants of the flush and 7 of the post type at strategic points. There is also a connection on the water-front for fireboat use.

Normally, the water supply is secured from a 16-in. connection with the mains of the Peoples Water Company, and the pipes are thus kept filled with fresh water, the necessary pressure being secured

by fire engines. In case of a large fire this fresh-water connection is closed, and salt water is pumped into the pipes from a pumping plant in the City Park, on the shores of Lake Merritt, with an intake extending far enough into its waters to insure an ample supply at all times. This lake, with a water surface of about 170 acres, is the extreme end of a salt-water estuary which extends from San Francisco Bay almost into the heart of Oakland. It has been cut off from the remainder of the estuary by a dam carrying a city street, under which large pipes carry the salt water back and forth as the tide ebbs and flows. Mr. Robson.

The pumping equipment consists of two 250-h.p. gas engines, each geared to a 4-stage turbine pump having a capacity of 1 000 gal. per min. against a discharge pressure of 200 lb. per sq. in. The pump-house is of reinforced concrete, designed to harmonize with its park surroundings.

Efforts Toward Municipal Ownership.

There have been efforts—more or less spasmodic—toward municipal ownership of the water supply by the Cities of Alameda and Contra Costa Counties for many years; until very recently, however, Oakland was the only one that had taken definite steps.

In 1874, a special committee of the Oakland Council made a report on the local sources of water supply. It is interesting to note that, even as early as this, the merit of the Calaveras Creek dam site and water-shed was recognized clearly. The properties of the Contra Costa Water-Works were also investigated and found to promise an adequate supply. The committee recommended that the city immediately secure its own water system. Shortly thereafter, the president of the Contra Costa Water-Works proposed two alternatives to the city: First, that its plant be bought on a valuation to be set by five appraisers—two from the company, and two from the city, the four to choose a fifth. The value found by a majority of the five to be binding; or, second, that the city lend its credit to the Water Company to an amount sufficient to complete San Leandro Dam and connect it by pipes with the mains in the city—at that time 8 miles distant. An option to buy the property at the end of 10 years at a value fixed as stated would also be given. Nothing was done by the city in the matter, and thus Oakland's opportunity was lost.

The city continued to grow rapidly, and the water problem was always a source of discussion, but little or nothing was done until in March, 1900, the Company began its famous suit over rates before Judge Hart who decided that the properties of the Company were then worth $7 000 000 and allowed 7% interest, 2% operating expense, and 1% maintenance. This allowance caused a raise in rates, and

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many private wells were the result. The suit cost the city $32 000 and doubtless cost the Company as much.

In 1902, Rudolph Hering, M. Am. Soc. C. E., made a report for a citizens' committee of Oakland. He advised that 15 000 000 gal. per day could be drawn from the Niles cones for the city's use.

In 1903, Desmond FitzGerald, Past-President, Am. Soc. C. E., reported to the same committee, and the next year a Board of Engineers consisting of A. M. Hunt and J. M. Howells, Members, Am. Soc. C. E., and Mr. F. C. Turner, reported to the city authorities on the adequacy of the Bay Cities Water Company's project for Oakland's water supply.

The report included a house-to-house canvass, with the following results:

Total number of private wells	718
Buildings supplied	1 436
People supplied	7 180
Water produced per day	718 000 gal.
Total capacity of tanks	3 467 000 gal.
Maximum and minimum depth of wells	681 to 18 ft.
General average depth	100 ft.
Population supplied	10 per cent.

Doubtless influenced by these reports, by the activities of the Bay Cities Water Company, which was threatening to invade its territory, and by the controversy with the city over rates, the Contra Costa Water Company offered either to sell or lease to the city, at a value determined by two men who should select a third if necessary. The city took no steps, and the offer was finally withdrawn when rates were fixed on a valuation of $4 700 000, the city ignoring the decision of Judge Hart giving a value of $7 000 000. This case was then on appeal to the Supreme Court.

In June, 1904, Judge Gilbert granted an injunction against the rates fixed by the city, but required the Company to file a bond for $130 000.

In October, 1904, five water supply propositions were submitted to the city, chiefly by the Bay Cities Water Company which was making every endeavor to break into the territory; and in March, 1905, an election was held on an offer submitted by the Bay Cities Company. The campaign will long be remembered as the hottest and most spectacular known in Oakland. A large sum of money is said to have been spent by the opposing water companies. The vote resulted in 4 369 for and 5 207 against the proposition.

Since that time, the water problem has been before the people continuously. The suit started in 1904 has not yet been decided. The Contra Costa Water Company has merged with other companies,

until now the corporation is known as the "Peoples Water Company". The Bay Cities Water Company has surrendered its activities to the Union Water Company. Mr. Robson.

The population has grown steadily, and the several communities on the East Shore now have the same water problem confronting them that Oakland left unsolved in 1874.

Financial and physical conditions make it almost impossible for one community to secure an adequate water supply without joining with others. This has been long realized, but the laws of California made no adequate provision for help. In 1909, a law following more or less closely the one under which the Metropolitan Water and Sewerage Districts of Boston and New York are operating, was passed by the State Legislature, but nothing has been done to organize a district because of alleged defects. At the request of the smaller communities, therefore, it was modified in 1911 to give them more representation. The present law permits any number of municipalities within the same county, as well as unincorporated territory, to unite and form a Municipal Water District.

In 1912, petitions were prepared and circulated to form a district, under the law of 1911, consisting of the municipalities of San Leandro, Alameda, Oakland, Emeryville, Piedmont, Albany, and some unincorporated territory, all in Alameda County. Sufficient signatures were finally secured, and it was thought a district would be formed. However, the Supervisors of Alameda County, who had the power to call the election, kept postponing the matter for one reason and another, until the present year, when an election was called for June. The Peoples Water Company put the stock control of the company, together with an option to sell its entire properties, the price to be fixed by the State Railroad Commission, in the hands of an Oakland bank.

Strong opposition to forming the district immediately appeared, a campaign of vituperation by the opponents followed, and attacks were made on the law, the water company, and the supporters of the measure. It seemed impossible to convince the electorate of the gravity of the situation, and even the majority of the newspapers were only lukewarm or actively opposed.

Due to the strenuous efforts of Mayor Heywood, of Berkeley, a majority of some 300 or 400 was given in favor of the district there, but this only nullified the adverse vote in the smaller communities, as Oakland returned a majority against the district of about 2 500. Less than 50% of the registered vote was cast. This defeat was a body blow to those who knew the conditions. Nothing has been done since, and it is not known what steps can or will be taken. A new law has been prepared, but should it be passed there will doubtless be as much opposition as before.

Mr. Robson.

In the meantime, the Peoples Water Company was becoming more involved, financially, and in July, 1914, interest on its bonded indebtedness was defaulted. The affairs of the Company have been put in the hands of a committee of bankers, and protective committees of bondholders have been formed. No satisfactory reorganization plans have been given out, and the general financial conditions resulting from the war in Europe leave little of an encouraging nature apparent in the immediate future.

Conclusion.

No satisfactory solution of the water supply problem for the East Shore cities can be reached until these can be joined in one district or organization for that purpose. When this has been accomplished, such properties of the Peoples Water Company as are necessary must be purchased and the development of local sources completed. The district would then be in a position to purchase and distribute water from any source available.

Half of the total present and proposed supply of the Spring Valley Water Company comes from Alameda County sources. Under existing laws, if San Francisco purchases the Spring Valley Company, the taxes now levied by Alameda County on this property would not have to be paid by San Francisco. This would mean a large decrease in revenue for general county purposes.

After forming a district and purchasing the properties of the Peoples Water Company, the East Shore cities should buy the Alameda County system of the Spring Valley Company, thus relieving San Francisco of this portion of the purchase cost and making available a larger sum for acquiring distant mountain supplies.

It is a striking commentary on the foresight of the Spring Valley Company, when it is known that for years it has had a deeded right of way from its Calaveras Dam to the City of Oakland.

Mr. Le Conte.

L. J. Le Conte,* M. Am. Soc. C. E. (by letter).—The writer is greatly interested in this timely paper. The author has certainly taken a very wide grasp of the whole complex situation, and in arriving at his conclusions has given due weight to all the important factors which influence a wise decision.

In comparing several schemes in which the construction cost is nearly the same, all the minor items—cost of operation, maintenance, depreciation, renewals, and incidental incomes—of course, are factors; but, when the first cost of construction is vastly greater in one case than in another, consideration of these smaller items can be omitted as they have no bearing whatever on the conclusion drawn.

Undoubtedly, the wisest policy for San Francisco is to develop, to the fullest extent possible, local supplies which will probably amount

* Berkeley, Cal.

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to 200 000 000 gal. per day, and be sufficient for 80 years to come. The city should then develop the pumping works at Moore's Landing on Old River, San Joaquin, and force the water up to Altamont Reservoir —capacity 1 000 000 000 gal. Thence supply mains should lead to the extensive natural gravel filter beds in Li ermore Valley, and thence to the several large storage reservoirs in the Alameda Creek water-shed.

The first development will be for 50 000 000 gal. per day, which will be sufficient for 25 years more, or, say, until 2000 A. D. The cost of these new developments up to this time will be:

Peninsula	$40 000 000
Pumping works	10 000 000
Total	$50 000 000

thus making a unit cost for development of $200 000 per million gallons per day. Moreover, distant future expansions of the supply can be readily made at the same unit cost for development.

The East Bay region, by full development of all local sources, can obtain from 30 000 000 to 40 000 000 gal. per day, which will be sufficient for from 10 to 15 years to come. At the close of this period—1930—the best available outside source will be, likewise, at Old River, just above the junction of Indian Slough, some 12 miles above Marsh's Landing, which will be the site of the pumping works.

Thence, two supply mains will lead along the straits and bay shore to Berkeley, Oakland, and Alameda. The first development should be for 100 000 000 gal. per day, which would call for the expenditure of $20 000 000. This additional supply would be sufficient for 60 years to come, or until 1975, and would show a unit cost of development of $225 800 per million gallons per day, delivered; and, moreover, the expansion of the supply, in the more distant future, can be readily made at the unit cost of $200 000 per million gallons per day.

These figures show conclusively that the distant Sierra supplies are undesirable, from every point of view, as will now be demonstrated.

In 1901, Mr. Grunsky reported that the Hetch Hetchy scheme, developed to 60 000 000 gal. per day, would cost $32 000 000, or $533 000 per million gallons per day, or nearly three times the cost of development of the local supplies near San Francisco. Likewise, in 1912, Mr. Freeman reported, with reference to the Hetch Hetchy scheme, that the first development for 160 000 000 gal. per day would cost (revised estimate) $64 000 000, showing a unit cost of development of fully $400 000 per million gallons per day. It would finally appear that this distant source of supply is vastly more expensive and is, therefore, undesirable.

Delta Sources.—The combined low-water flow of all the rivers naturally emptying into Carquinez Straits is now approximately 7 200

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cu. ft. per sec., during the dryest seasons on record. This is simply the surface or visible flow, and does not include the underground supply, which would probably increase it materially, if it were known; but, even the minimum (taken at 7 200), will give a reliable supply of 4 615 000 000 gal. per day. Now, it is a well-established fact that the large future hydraulic developments in the great Central Valley of California—in the way of irrigation, power, flood control, and extensive reclamation and irrigation works at and near Tulare Lake—will collectively tend to increase materially the low-water flow of all the rivers in the delta region.

The great works at Tulare Lake and vicinity will certainly add practically 10 000 sq. miles to the direct water-shed of the San Joaquin proper, and will certainly add 2 000 cu. ft. per sec. to the low-water flow of the river at a point just above Firebaughs. This result, combined with the underground return flow from the great irrigation fields on both sides of the river, will ultimately swell the low-water flow of the San Joaquin River to fully 6 000 cu. ft. per sec., thus greatly improving the low-water steamboat navigation on the entire river, to the everlasting benefit of the public in general.

The same beneficent results will also take place on the Sacramento River, and the writer is of the opinion, therefore, that fully 10 000 000 000 gal. per day can be considered as the reliable delta supply in the distant future.

Of course, subsidence and filtration, sooner or later, are to be expected for all supplies taken from the delta; but, naturally, this work is reduced to a minimum by placing the intakes on the south or Old River side, where long experience shows turbidity to be a minimum.

The quantity proposed to be taken from the delta supply, in the near future, is only 150 000 000 gal. per day, or 3% of the total, hence it can safely be said that there will be no trouble whatever from salt water.

In conclusion, it would appear that San Francisco and the East Bay region can both safely lay aside the idea of calling on the distant Sierra supplies for the next 100 years; and they will certainly be far better off financially by doing so.

Mr. Grunsky.

C. E. GRUNSKY,* M. AM. SOC. C. E. (by letter).—In Mr. Cory's presentation of this subject, which is of vital importance to a flourishing and fast growing western section, there is a marked departure from the ordinary treatment by an engineer of an engineering problem. He has endeavored to present a picture of the local circumstances of growth and development of San Francisco Bay communities and of the factors which influence this growth, in order that due weight may

* San Francisco, Cal.

Mr. Grunsky.

be given to social, industrial, agricultural, topographic, and hydrographic conditions when the question of a water supply for the Greater San Francisco of the future is considered. The result is a paper full of interest to the California engineer, but with only few points that may call forth some comment from the engineer who deals with water supplies from the eastern standpoint.

It is plain that the author favors the development of local sources of water supply to the limit, before going far afield for a mountain supply of a higher grade.

The established company, the Spring Valley Water Company, formerly known as the Spring Valley Water-Works, has a developed supply of 35 000 000 gal. per day. This company is at present supplying in excess of this quantity, but at the cost of a gradual depletion of its storage. In California the rain which produces run-off falls within the consecutive 6 months, November to April. There may be from 2 to 3 years in succession with only 50%, or but little more than 50%, of normal rain.

In the water-sheds from which San Francisco's present supply is obtained, the normal rainfall ranges from about 15 to 50 in. per annum (the latter covering a few square miles only), and, in the years in which the rainfall is but little in excess of the minimum, which may be as low as 40% of the normal, there is hardly any run-off. Due to these conditions, there is need for storage, either on the surface or underground, of practically a 3 years' supply, if the local sources alone are to be made dependable.

Due to this fact, the supply from near-by sources involves, first, large reservoir capacity, and second, in the absence of filtration, the control of vast areas of water-shed, in order that by such ownership adequate sanitary protection can be afforded. The Water Company, in line with these requirements, has constructed reservoirs on the peninsula south of San Francisco, which have an aggregate capacity of about 28 000 000 000 gal., and is now constructing an additional reservoir on the opposite side of the bay, on Calaveras Creek, a tributary of Alameda Creek, which will more than double this capacity. The Water Company, furthermore, in order to protect the quality of the water, and also to safeguard rights to the underground waters of the Livermore Valley, has acquired and holds the fee to more than 100 000 acres of land, much of which, by reason of use as water-shed or as reservoir area, is made non-available for agricultural and residence purposes. The collecting ground is but slightly removed from the bay margin, and is generally of the low rolling hill type. Nearly if not quite one-half of the present San Francisco water supply comes from wells in the Livermore Valley and from the gravels in Sunol Valley. The Sunol gravels are recipients of the summer flow of Calaveras Creek and of lesser feeders of Alameda Creek, and in

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a small measure afford sub-surface storage. The water is collected from the gravels in a filter gallery.

The wells in the Livermore Valley near Pleasanton were originally Artesian, but the water-table has been drawn down by pumping, so that all water obtained from this source is now pumped.

The combined supply from the Pleasanton wells and from the Sunol gravels, after being carried across the Bay of San Francisco from the Alameda to the peninsula side, is pumped into the Crystal Springs main, so as to arrive in the city at an elevation of about 165 ft. Certain quantities of this water, according to requirement, are pumped to higher levels.

The water intercepted at Pleasanton and at Sunol is that which under natural conditions would have aided in keeping the Niles gravel cone, the delta cone of Alameda Creek, supplied with water. By this interception the natural period of flow of the creek has been reduced, and in some measure the sub-surface waters of a fan-shaped area expanding with distance westward from Niles toward San Francisco Bay have been reduced. This sub-surface supply is drawn upon, as Mr. Cory explains in his paper, by the pumps at Alvarado and elsewhere which supplement the Lake Chabot and other local supplies of Oakland, Alameda, and Berkeley.

It seems probable that the gravels of the Alameda Creek delta cone are being already drawn upon for more water than they can continuously supply. The water-plane near the pumps is lowered below sea level. It remains to be seen how long the draft of the past upon such wells can be maintained without impairing the quality of the water beyond the allowable limit.

The possibility exists, as Mr. Cory points out, of making additions to the developed local supplies, particularly for the cities on the east shore of the bay; but this water development, as in the case of the San Francisco supply, implies the acquisition of large areas of water-shed and of near-by reservoir sites, and, in view of the low, undesirable nature of the collecting ground, ultimate filtration. All this will be at a cost not materially, if at all, less than that of first-class mountain water.

The fact is generally recognized that the population centering around San Francisco Bay cannot depend long on the water from the so-called near-by sources, but must soon go to some remote source for an adequate supply.

The question naturally arises: Which of the remote sources will be the most available? Pumping from the San Joaquin or from the Sacramento River has frequently been advocated, and much is made of the fact that inferior water to that which these streams could provide is made fit for consumption elsewhere by filtration and by treatment with hypochlorite.

Mr. Grunsky.

Although this may be true, no one who has the choice would advocate the preparation of a filthy water for domestic use when, within the limit of the means at command, a pure mountain water can be made available.

On the basis of an exhaustive analysis, the writer reached the conclusion in 1901 that the cost of developing and delivering a Sierra Nevada supply of water would not be greatly in excess of developing and delivering an equivalent quantity from local sources. The result of his investigations is embodied in a so-called progress report published in the Municipal Records of San Francisco, and his conclusions have been confirmed, particularly in so far as the selection of Tuolumne River as the best source of supply is concerned, by subsequent investigators for the City of San Francisco.

If the city acquires the properties of the Spring Valley Water Company, at the price at which they are to be shortly submitted, it will be at a cost per million gallons of developed water with which the cost of water from the Sierra Nevada, if brought to the bay region in a quantity not less than 60 000 000 gal. per day, would compare favorably.

It would be out of place to attempt a review of the studies which led the writer to make the recommendation that the city make application to the Secretary of the Interior for the right to store waste flood-waters in reservoirs to be constructed at Lake Eleanor and at the Hetch Hetchy Valley, the latter on the main branch of the Tuolumne River; but, though making this definite recommendation and thereupon making the application for the city, through the Mayor, the writer has always been a strong advocate of the acquisition by the city of the established water-works, to which the mountain source of supply would be added.

The writer, as City Engineer of San Francisco from 1900 to 1904, was required to report on a water-works entirely independent of the established system, but, in doing so, he called attention to the proper course to be taken by the city. It is needless to say that he has been disappointed in the result of subsequent proceedings.

The city's application for reservoir sites met with opposition which was directed by Spring Valley Water Company officials. These officials succeeded in alarming the two irrigation districts which were dependent on the Tuolumne River as their source of supply, and thereafter the Water Company, the two irrigation districts, and misguided Nature lovers, actively opposed the granting of permits to San Francisco to use the two selected storage sites.

Secretary of the Interior Hitchcock advised that Congressional action in the matter should be sought. His successor, Secretary Garfield, recognized San Francisco's rights, but made the permit which he issued subject to certain conditions, the most onerous of which

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was the one requiring that the lesser reservoir, Lake Eleanor, be developed first. As the permit was revocable, the next Secretary of the Interior, Mr. Ballinger, unfriendly to the San Francisco project, issued an order to San Francisco to show cause why the permit, in so far as it related to the Hetch Hetchy Valley, should not be revoked.

San Francisco was thus put on the defensive and, due to lack of confidence of various of her official departments in each other, placed this case in the hands of an expert called in from the East. The natural result was that thousands of dollars were needlessly expended in the accumulation of a mass of statistical information, and plans were outlined, not for the San Francisco water supply alone, but for a project to divert from the Tuolumne River more than 600 cu. ft. per sec., and to make use of this water until such time as the cities near the bay require it, for irrigation in the bay region.

It was pointed out in the resulting reports that the earlier project, for an initial unit of 60 000 000 gal. per day, was far below the yield of Tuolumne River, and that the city should undertake the larger project in order to perfect its claim to a larger quantity of water. No assent, of San Francisco or of any of the other cities supposed to be interested in the larger project, has yet been obtained, or has ever been asked for.

This ambitious project when presented to the Secretary of the Interior led to violent opposition on the part of the owners of land naturally irrigable from the Tuolumne River. These land owners were determined that, if any surplus water in excess of the needs of San Francisco were to be used for irrigation, it should not be used on the other side of the Coast Range.

The representatives of the irrigation interests were therefore in Washington at the hearings, and they neatly accomplished the discomfiture of San Francisco. By claiming that the river would not always afford water enough both for irrigation and the municipal requirements of the Bay Cities, 400 000 000 gal. per day according to the tentative project, the city's experts were put to it to show that the water supply was ample. They did this, at least to their own satisfaction, and then the demand was made by the irrigation districts that, this being so, the permit if granted should be conditioned on the irrigation districts first receiving all the water which they could use beneficially before San Francisco takes any, and that, to the extent necessary to protect the districts in such use, they should be allowed to take San Francisco's stored water at cost.

Instead of refusing to surrender first right to all stored water, the city fell into the trap, as can be seen by reference to the "Raker Law" cited by the author, under which certain rights were granted to San Francisco.

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The city's representatives at Washington should have insisted on her prior rights. There should have been no concession of rights in the stored water to the irrigationists. The courage to do this was lacking. At the same time there might well have been, as the writer has always advocated, a cordial co-operation with the irrigation districts, because the city, by storing much more water than called for by its immediate needs, would have been in position for many years in the future, probably for 25, or more, to give annually large volumes of stored water free of cost to the districts, thereby saving to them the need of increasing their fall supply by storage of their own.

San Francisco has apparently now waived her prior rights to the stored flood-water of Tuolumne River. When there is a shortage of supply, it will fall on San Francisco, not on the irrigation districts, which, except for the concession made, as explained, would have had no rights whatever in the water stored by San Francisco, provided, of course, that this storage was made subject to the acknowledged prior rights of the districts to the natural flow of the stream to the limit of these prior rights.

San Francisco, under the Raker Law, is to expend from $500 000 to $1 000 000 on the proposed storage works in excess of her own requirements in order to be in a better position to meet the obligation, which she has apparently assumed, to supply some stored water to the irrigation districts.

San Francisco, under the same law, will be required to pay for the appropriated water, which should be hers without cost, under State law, at the rate of $15 000 per year for 10 years beginning in 1919, thereafter $20 000 per year for 10 years, and thereafter $30 000 per year. The present value of these payments exceeds $300 000.

San Francisco must pay excessively for the Government land occupied by its dam sites, by its reservoirs, and by its tunnels, conduits, power-houses, transmission lines, and the like. For these lands, of little or no value (the city already owns the fee of the major portion of the level land in Hetch Hetchy Valley), there must be given in exchange lands of greater value acquired by the city but not necessary in connection with the project.

San Francisco, at the direction of the Secretary of the Interior, must construct roads and trails in the National Forest Reservation in which Lake Eleanor and the Hetch Hetchy Valley are situated, at a cost of from $500 000 to $1 000 000, and must provide for the maintenance thereof.

San Francisco is compelled, too, to develop power and to sell a large part of this power at cost in Modesto and Turlock irrigation districts. Perhaps the cost will be determined so as to include a part of the storage, perhaps not, because storage was essential for the other purposes. The sale of power at cost will be of little if any

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benefit to the San Francisco water-rate payer. The remaining power will be available for use in San Francisco. Its transmission will be over lines about 150 miles in length. It would probably cost less to develop the same power in San Francisco with oil at 75 cents per bbl. than to develop the water power and deliver it over electric wires. Consequently, although of economic value to society, the power feature is likely to prove a burden, and not an advantage, to the project as a whole.

San Francisco has voted bonds for the construction of a 60 000 000-gal. per day project. The bonds voted amount to $45 000 000. These bonds are not available for the new 400 000 000-gal. per day project based on conditional rights to water and on the requirement that a large part thereof be spent in a commercial venture involving the sale of power. The city is proceeding, however, in the development of the power as though its rights in the matter, particularly in reference to the use of this bond money, could not be questioned.

The Hetch Hetchy bonds were voted in November, 1909. They cover a project, as stated, for a supply of 60 000 000 gal. per day to be obtained from Lake Eleanor under the Garfield permit. They cover also an expenditure possibly of $7 000 000 or $8 000 000 for a distributing system of reservoirs and pipes in San Francisco. They were voted with the understanding, so expressed in the ballot, that, if the purchase of the Spring Valley Water Company's properties in their entirety were sanctioned at the same time by the voters, only $30 000 000 would be available for the works necessary to make the Tuolumne project the auxiliary supply to the established system. The purchase of the Spring Valley system for $35 000 000 at the same election was lost by a few votes short of the necessary two-thirds.

At the time the city thus voted bonds, there should have been created a Board of Water Supply, free from politics. This was not done. The administration at that time and the administrations that have followed, would not let go. The Mayor, who names the Board of Public Works, and who practically selects the City Engineer for the Board of Public Works, the Supervisors, with the Public Service and Finance Committees, the City Attorney, the Board of Public Works, and the City Engineer, all have a hand in the game.

There is no programme as yet for any change. Within a year there was named a Board of Consulting Engineers to pass upon the hydro-electric features of the first power installation under the unauthorized project, understood to be for 50 000 h.p. These engineers were designated by name in the City Engineer's request for their appointment. From this circumstance alone, it is to be inferred that the political management of the city's water project will be continued. Deplorable as this may seem, it is yet possible that competent management will not be too greatly hampered by political

obstruction and political direction; and the hope may be expressed that, despite the unfortunate circumstances of the past, San Francisco may yet achieve an adequate and in every way satisfactory water supply. Nevertheless, the danger should be recognized that competent management of to-day under the political system may be followed by incompetent or even corrupt management to-morrow. Mr. Grunsky.

It will be seen from the foregoing that the Hetch Hetchy project under the Raker Law is not the writer's Hetch Hetchy project, and that the new project for water for the entire bay region has not yet been given proper standing by the voters of San Francisco, nor has it yet received the necessary study to determine its main structural features.

Meanwhile the city has made endeavor to buy the established waterworks. According to the latest proposition, to be voted on some time next spring, the Water Company will be allowed to retain several thousand acres of valuable land in and adjoining San Francisco, besides some 5 000 acres of land in Livermore Valley. The value of the Pleasanton wells as a part of the system will be reduced because the maximum draft on them is to be limited to 15 000 000 gal. per day, and because the ownership of the 5 000 acres will carry with it the right to draw on the Livermore gravels for all the water that may be needed on these lands. As these lands are fertile and lie in a region with a mean annual rainfall of only 15 in., the demand for water by them will in time be such that there may not be much water left in the gravels for San Francisco. It seems unfortunate that in the agreement for purchase the limit to 15 000 000 gal. in a single day has been made, because in the future, when San Francisco has a gravity mountain supply, the pumped water from Pleasanton wells will have chief value as an emergency supply, and when used in an emergency there should be no restriction on the quantity to be taken in a single day.

The author discusses the local water supplies as though their development, in advance of a good mountain supply of unquestionably superior quality, was a matter of course. If this is his view, there are others who are not convinced that, even on the score of cheapness alone, the development of the local sources first can be justified, and, when quality, quantity, cost, and reliability of service are all taken into account, it is hard to see how any other conclusion can be reached than that the mountain source is the next one which should be developed. There should be 60 000 000 gal. per day added to the supply at present available, and this should be mountain water.

In summarizing the estimate of the total ultimate safe draft for municipal purposes on near-by sources of supply, the author combines the developed and the undeveloped supplies, referring to the aggregate as obtainable at a reasonable cost.

Mr. Grunsky.

He includes as additional sources all or in part, though perhaps not at the estimates noted in Table 6, which are those of Messrs. Chittenden and Powell.

TABLE 6.

The Peninsula Coast streams, at	50 000 000	gal. per day.
" Ravenswood and Alviso Wells, at	20 000 000	" " "
" Calaveras and San Antonio Branches of Alameda Creek, at	90 000 000	" " "
" Area tributary to Livermore Valley, at	40 000 000	" " "
Total	200 000 000	" " "

Of this quantity (which no doubt includes some over-estimate), only about 15 000 000 gal. per day are actually developed. In the development of the remainder, as already stated, the cost—per million gallons of daily supply—may be as much as, or more than, that of the water from the Tuolumne River, if brought down in a first unit of at least 60 000 000 gal. per day. Some of this remainder will not be obtainable except at the expense of other communities or at a cost to society, which is as necessary to consider as that charged by the author to a Sierra supply.

He includes also from 12 000 000 to 13 000 000 gal. as being furnished by pumping from the Niles gravel cone, and is right in questioning the permanency of a draft of this quantity, and he includes a large development from small creeks in the hills to the east of Oakland, aggregating 21 000 000 gal. per day, and at present developed to but a small extent by the San Leandro Reservoir. When the cost of filtering is considered, and the cost to society, by reason of the occupation of valuable territory, these sources will probably be found to be as expensive as an adequate Sierra supply.

Whether or not this is actually the case, the fact remains that a 60 000 000-gal. first unit from the Tuolumne is within reach, from the standpoint of cost. The development of additional local sources is to be discouraged and not encouraged. The high-grade mountain water should be made available before the possible but improbable development of the nearer sources is awaited.

The writer's conviction remains unshaken that the private concerns on both sides of the Bay of San Francisco should be bought out by the people, and that the entire bay community should then join in perfecting and carrying out as speedily as possible the Tuolumne project, but with suitable modification of the Congressional permit, which is unfortunate not alone for San Francisco, but also for all Western States, in this, that it establishes a dangerous precedent.

The water supply problems to the northward of San Francisco Bay are separate and distinct from those of San Francisco and the East

Bay cities, and need not be brought under discussion in connection with the larger San Francisco problem. Mr. Grunsky.

The author presents a diagram, Fig. 15, supposed to represent the latest information relating to rain distribution in central portions of California. This should be used with caution. The precipitation does not increase from north to south in the Sierras, as this diagram indicates, and there is no point known to the writer where, south of the latitude of San Francisco, the precipitation on the western slopes of the Sierra Nevada approaches 80 in. per annum, as shown by the curves of this diagram.

The same suggestion applies to the run-off diagram, Fig. 16. Dependence on the curves there shown, particularly for the Sierra Nevada eastward from San Francisco, might lead to disappointment.

As a contribution to the rainfall and run-off information in the San Francisco Bay region there is submitted herewith the result of a study made by the writer several years ago.

The run-off curves on Plate I will show at a glance the limit of the water crop at any point under consideration. Where, as in the case of the Crystal Springs reservoir of the Spring Valley Water Company, the storage capacity is so large that practically no water goes to waste, the run-off shown by the curves, less the loss from the reservoir by evaporation and seepage, will determine its productiveness. The curves afford a quick first approximation of the possible limit of yield.

The frequency of years of heavy, moderate, and light rainfall in the vicinity of San Francisco are shown by Fig. 27. The rainfall is expressed as a percentage of the normal annual rain in order to give the diagram wider use than just for the locality for which prepared. These data can readily be fitted into a probability diagram to obtain departure from the ordinary law by any one interested in this climatic peculiarity.

*The writer is sorry that what he stated in the foregoing discussion should have been construed as a reflection either on the course taken by the City Engineer of San Francisco in calling for a board of consulting engineers, or by these engineers who are defending themselves against a fancied imputation of being political engineers. The writer has been on the best of terms with each of these engineers; they have his unqualified respect and confidence; and he would not intentionally have said anything to their discredit. His only purpose in referring to the appointment of consulting engineers was to show that no intention was apparent on the part of the present administration to create a Water Board, and that the present plan of managing municipally owned utilities would probably be continued.

* The following part of Mr. Grunsky's discussion was written after reading Mr. O'Shaughnessy's discussion.

Mr. Grunsky.

The statements from the writer's earlier discussion, which are objected to, and which have been quoted, should be read in connection with the following which was a part thereof:

"Nevertheless, the danger should be recognized that competent management of to-day under the political system may be followed by incompetent or even corrupt management to-morrow."

Neither in this statement, nor in a preceding use of the term "political", is there any intent to cast an unfavorable reflection on those who happen to occupy official positions. The instability of their tenure of office, however, is self-evident, and the large number of depart-

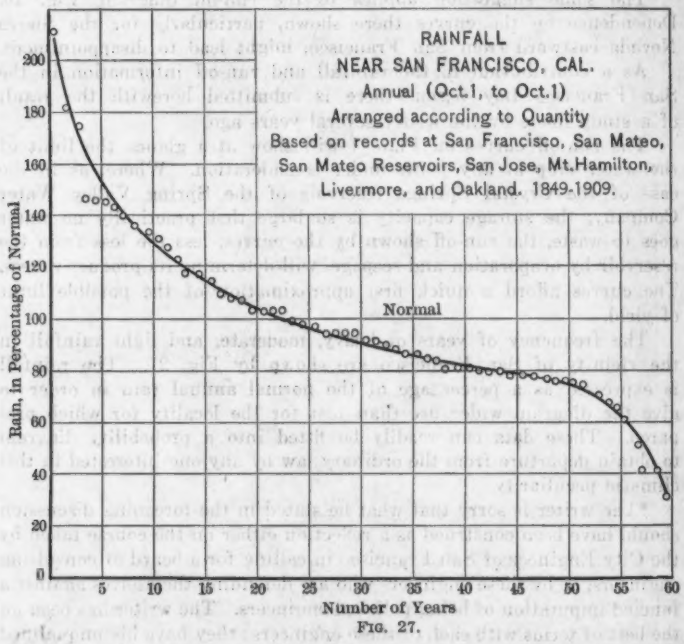
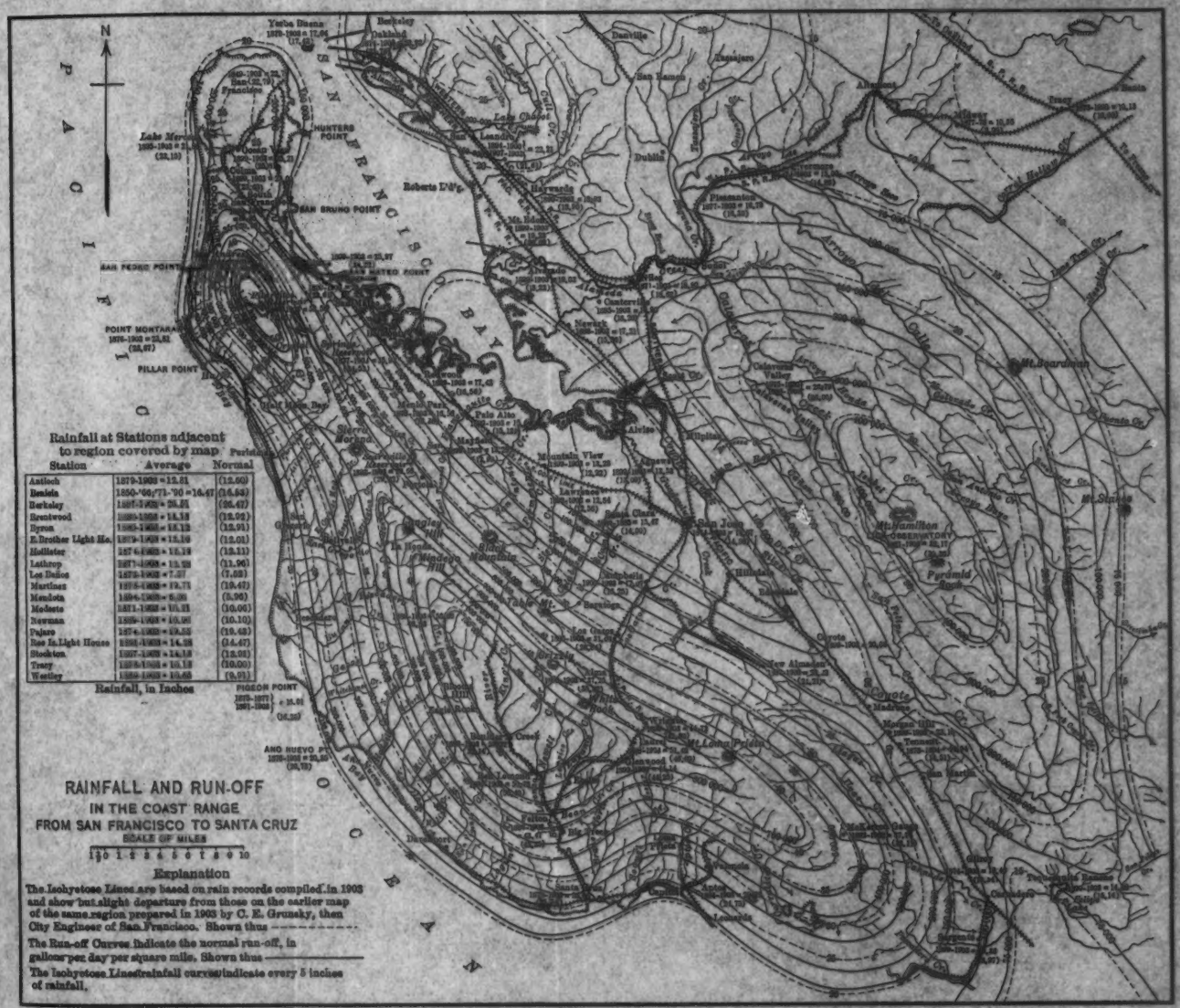


FIG. 27.

ments or sub-departments of the City's Government which have been and are assisting in directing the acquisition of a municipal water supply, and will have a voice in its management, condemn the system. No evidence is yet apparent that the creation of a stable Water Board is under contemplation, and it was to show this fact, and for this purpose only, that reference was made to the appointment of a board of engineers by the City at the request of the City Engineer. The writer agrees with those who state that it was better to have the con-


N
PACIFIC OCEAN
SAN FRANCISCO BAY
Yerba Buena 1870-1903 = 17.86 (17.42)
Berkeley
Oakland
Alameda
Point Bonita
Lake Merced 1895-1903 = 21.1 (22.13)
HUNTERS POINT
Lands End
Golden Gate
Presidio
San Francisco
Fort Point
Alcatraz
Angel Island
Point Blunt
Point Diablo
Point Tiburon
Sausalito
San Rafael
Corte Madera
Larkspur
Ross
Kentfield
Fairfax
San Anselmo
Lagunitas
Mt. Tamalpais
Mt. Tamalpais Summit
Mt. Tamalpais Station
Mill Valley
Sausalito
Tiburon
Belvedere
Point Richmond
Richmond
San Pablo
Pinole
Hercules
Crockett
Port Costa
Martinez
Benicia
Vallejo
Mare Island
Suisun
Fairfield
Cordelia
Napa
Napa Junction
Yountville
St. Helena
Calistoga
Petaluma
Sonoma
Glen Ellen
Kenwood
Santa Rosa
Healdsburg
Windsor
Cloverdale
Geyserville
Asti
Cazadero
Duncans Mills
Guerneville
Monte Rio
Occidental
Bodega
Bodega Head
Tomales
Point Reyes
Point Reyes Station
Olema
Inverness
Point Reyes Light
Point Bonita
Point Lobos
Fort Cronkhite
Fort Barry
Fort Baker
Fort Mason
Fort Scott
Fort Winfield Scott
Fort Point
Golden Gate
Lime Point
Point Diablo
Point Tiburon
Point Richmond
Point Pinole
Point Molate
Point Orient
Point Potrero
Point San Pedro
Point Montara
Pillar Point
San Pedro Point
San Bruno Point
San Mateo Point
Roberts Dd'g.
Burlingame
San Mateo
Redwood City
Menlo Park
Palo Alto
Mayfield
Stanford University
Mountain View
Los Altos
Los Altos Hills
Alviso
Milpitas
Berryessa
Alum Rock
Evergreen
Penitencia
Dublin
Pleasanton
San Ramon
Danville
Livermore
Arroyo Las Positas
Arroyo Mocho
Arroyo Valle
Arroyo Seco
Arroyo Del Valle
Arroyo Del Hambre
Arroyo Del Cierbo
Arroyo Del Puerto
Arroyo Del Rincon
Arroyo Del Agua Caliente
Arroyo Del Agua Puerca
Arroyo Del Agua Fria
Arroyo Del Agua Dulce
Arroyo Del Agua Salada
Arroyo Del Agua Negra
Arroyo Del Agua Blanca
Arroyo Del Agua Verde
Arroyo Del Agua Roja
Arroyo Del Agua Amarilla
Arroyo Del Agua Azul
Arroyo Del Agua Morada
Arroyo Del Agua Rosada
Arroyo Del Agua Gris
Arroyo Del Agua Parda
Arroyo Del Agua Negra
Arroyo Del Agua Blanca
Arroyo Del Agua Verde
Arroyo Del Agua Roja
Arroyo Del Agua Amarilla
Arroyo Del Agua Azul
Arroyo Del Agua Morada
Arroyo Del Agua Rosada
Arroyo Del Agua Gris
Arroyo Del Agua Parda
Mt. Boardman
Mt. Hamilton
Mt. Diablo
Mt. Stakes
Pyramid Rock
Coyote
Madrone
Morgan Hill
Gilroy
Hollister
San Juan
Tres Pinos
Chittenden
Pajaro
Watsonville
Aromas
Salinas
Castroville
Moss Landing
Monterey
Pacific Grove
Point Pinos
Carmel
Point Lobos
Point Cypress
Point Joe
Point Pinos
Point Lobos
Point Cypress
Point Joe
Point Pinos
Point Lobos
Point Cypress
Point Joe
Point Pinos

sulting engineers selected by the City Engineer, who understands the problems to be passed on, than to have left their selection to other City officials. Mr. Grunsky.

The writer believes the *Transactions* should not be burdened with a reply to the personalities contained in the discussion by Mr. O'Shaughnessy.

WILLIAM SCHUYLER POST,* ASSOC. M. AM. SOC. C. E. (by letter).—This paper is an inspiration. It indicates to the engineer a legitimate field which he has abandoned and left to the cultivation of promoters and chambers of commerce. All have smiled at the vagaries of the western "booster", and admitted that people were fools to believe; but we have "stood for it", and have not realized that here is a real science of economic importance, one which the engineer is competent to pass on, and which he should pass on—the assigning of values to elements of civic growth, assigning weights to these values, and making a logical deduction from the known to the predicted. Mr. Post.

It is an ordinary platitude that a city must look 50 years ahead in its water supply. The simple matter of width of streets is left to the accident of original dedication. The cost of future congested traffic and the economic wastes of inadequate or over-adequate present construction and planning all belong to a group which invites the intellectual and imaginative qualifications of the engineer, wherein, working in the present, he foresees, provides for, and harmonizes, the economic situation a decade or more beyond.

As an Angeleno, and with a more or less profound geologic knowledge, the writer cannot admit that the San Francisco fault line runs near Los Angeles. In fact, he believes that the nearest active fault is 60 miles distant, and earthquake disturbance is in reality negligible in Los Angeles. However, the writer enjoys admitting that the eastern colonist puts all California in the earthquake class, and balances it in his mind with cyclones in the Middle West. Therefore, as far as the psychology of the colonist is concerned, before he starts for the coast, the conclusion of Reason 12 is sound; and therein lies the interest of such investigations as these.

M. M. O'SHAUGHNESSY,† M. AM. SOC. C. E. (by letter).—The writer has read this paper carefully and confesses himself embarrassed to analyze its motive and value. There has been much discussion in recent years as to the various fields in which the engineer may become active, outside of the exercise of his profession as such, and Mr. Cory's paper certainly invades many foreign subjects into which it has never been even suggested that the engineer should inject himself. Mr. O'Shaughnessy.

Mr. Cory enters into a discussion of political, sociological, moral, economic, and financial features of a great and varied State, and all

* San Diego, Cal.
† San Francisco, Cal.

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the sub-divisions thereof, and it is to be feared that he is so enthusiastic in those realms that he neglects the engineering side of the problem.

One of the earliest precepts instilled into the minds of young engineers, both before and after their entrance into the practice of their Profession, is the necessity of obtaining as exact information as possible on a given subject before proceeding with any independent work thereon. It is evident, however, that this precept has been, to some extent at least, disregarded in the present case, as some of the author's statements are only partly true, and others, so far as the knowledge of the writer goes, are not true at all.

The paper consists chiefly of a résumé of matters which were thoroughly and scientifically discussed and settled at the time the grant of the Hetch Hetchy reservoir sites, rights of way, etc., to the City of San Francisco was under investigation by Federal Government and other experts.

Mr. Cory attempts to revive a number of dead issues, and questions the engineering and financial policies of San Francisco as if these matters had not already been thoroughly considered and disposed of by a number of distinguished engineers, whose names are recognized throughout the United States—and throughout the civilized world—as authorities in their respective fields of work.

The Hetch Hetchy project has been the subject of a vast amount of painstaking thought and labor on the part of the officials of San Francisco and the experts whose advice was sought by them. The people of the City understand the situation of the municipality with regard to the water supply, and are as thoroughly in accord on the Hetch Hetchy scheme at the present time as they showed themselves to be some years ago, when propositions to bond the City for the acquisition of lands and for the construction of the works were carried by a vote of six to one.

The City has offered the neighboring East Bay communities—Oakland, Berkeley, Alameda, and the smaller cities—every opportunity to join with her in obtaining a mountain water supply. These cities naturally form a part of a Metropolitan District the interests of which will be best served in co-operating with the project of San Francisco. Up to the present time all the Bay cities have been furnished with water by private corporations. For years, the Spring Valley Water Company of San Francisco bitterly fought the acquirement by the City of rights in the Hetch Hetchy region, and by a skilfully planned propaganda created very strong opposition to the City among the irrigationists whose lands are contiguous to the Tuolumne River. This corporation opposition did not cease until the Raker Bill was passed.

The City officials have since agreed with the Company on a purchase price for its property, and this is to be submitted to the voters

for ratification at a special election in March, 1915. There is some opposition to the proposal, based on the idea that instead of buying the Spring Valley properties at a price which those opposing the plan consider excessive, the City should proceed at once to the construction of a Hetch Hetchy system, and rely entirely on this source, independent of the existing works of the Spring Valley. The opposition from the irrigation districts to the Hetch Hetchy scheme was modified on the inclusion of conditions, in the Raker Bill, which guarantee to the irrigationists their developed rights and a much larger quantity of water than they were technically entitled to under the State laws for 60 days of the flood period of each year; thus, there is no longer any serious opposition to the Hetch Hetchy project from any faction in San Francisco or in the San Joaquin Valley, except from some irreconcilable agitators. Mr. O'Shaughnessy.

The people of the transbay cities generally favor uniting with San Francisco to form a water district, and such opposition as now exists in those communities has been stirred up by the Peoples Water Company, which is in financial turmoil from imprudent high financing. This Company made futile opposition to the Raker Bill. Its only hope for survival is to keep its customers out of the proposed Metropolitan Water District.

Such is the general situation of the San Francisco Bay region with regard to the mountain water supply. The question of necessity has been settled; the Hetch Hetchy has been found to be the most available source of supply by a board of distinguished and disinterested army engineers, either for San Francisco alone, or for the proposed water district. San Francisco, without waiting for the formation of the Metropolitan Water District, has already begun construction work in the Hetch Hetchy region.

The writer cannot see in what respects the matters touched on by this paper are of sufficient interest to the engineering world to justify its publication by the Society; but, as it has been published, he will, without attempting to reply to it in detail, indicate a few points on which he takes issue with the author.

First, as to competition between the San Francisco Bay cities and Los Angeles: The Bay cities do not regard themselves as competitors of Los Angeles. This statement must not be taken as a confirmation of Mr. Cory's idea that San Francisco is self-sufficient. It means that San Francisco recognizes her southern neighbor's place in the general scheme of things; that the interests of the two cities are dissimilar; that the prosperity of one is helpful to the prosperity of the other; and that San Francisco realizes that increased prosperity in Southern California benefits the State as a whole.

However this may be, competition or non-competition between the Bay cities and Los Angeles could not be a factor in determining

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the choice of a water supply. Conditions are so different in the two sections of the State that a matter of such small import as the annual water bill could cause little difference to any one who is making his choice of a permanent home, or be a final factor in determining the moral status or commercial triumph of either city. The comparison of sociological conditions in the two communities is no part of an engineering problem. However, personal knowledge and civic pride impels the writer to correct a few statements in Mr. Cory's comparison, which are either wholly or partly incorrect.

In a comparison of growth, on page 8, the growth of the San Francisco Metropolitan District from 1900 to 1910 is given as 41 per cent. Referring to the Abstract of the 1910 Census, page 62, it is found that the figure should be 45.2 per cent. A comparison of the population curves, Fig. 2, with the figures of the Census on page 62 of the Abstract, shows that in Fig. 2 the population of "Greater San Francisco-Oakland" in 1900 is 25 000 less than the Census figure, and in 1910, 60 000 less; the population of "Greater Los Angeles" in 1910 as plotted is 30 000 less than the Census figure. These discrepancies are not very large, but they work against the northern cities. Furthermore, any comparison between the growth of San Francisco and that of Los Angeles since 1900 must take into account the retardation of San Francisco's growth by the disaster of 1906 and the great apparent acceleration of that of Los Angeles due to the annexation of a number of small cities and towns in the outlying districts.

On page 17, it is stated that the San Francisco port charges and regulations are onerous. Referring to a paper by H. M. Chittenden, M. Am. Soc. C. E., entitled "Ports of the Pacific",* the writer would call attention to Table 6, "Comparative Costs to Ship and Cargo in Pacific Coast Ports", and quote from the note at the end of that table:

"This table is the result of a comparative study of the rates in the different ports with a view of determining the relative tax on commerce in each. In order to arrive at a condensed approximation of the cost of terminal charges and a comparison of the principal Pacific Coast ports, a purely hypothetic case has been assumed of a 6 000 net ton register ship, 30-ft. draft, discharging and loading cargoes of 12 000 tons of miscellaneous freight, the ship remaining in port 12 days."

The total cost to ship and cargo in the various ports is shown in Table 7, from which it will be seen that the cost paid by the ship is three times as great at San Francisco as at Los Angeles; the cost paid by the cargo at San Francisco is 79% of that at Los Angeles; and the cost paid by the ship is so small a part of the total cost that the total cost at San Francisco is less than 84% of that at Los Angeles.

* *Transactions*, Am. Soc. C. E., Vol. LXXVI, p. 208.

Furthermore, Table 7 shows the total cost at San Francisco to be lower than at any of the other five ports mentioned. It is immaterial to both the shipper and the ultimate consumer how the port charges are divided, for that which interests them is the total amount they have to pay. Mr. O'Shaughnessy.

TABLE 7.

	Total cost.	Cost paid by ship.	Cost paid by cargo.
San Francisco	$18 042.50	$1 242	$16 800.50
Los Angeles	21 599.00	407	21 192.00
San Diego	22 475.00	875	21 600.00
Portland	22 280.00	680	21 600.00
Puget Sound	21 800.00	200	21 600.00
Vancouver, B. C.	21 830.00	230	21 600.00

The term, "Paris of America", applied by the author to San Francisco, is an echo from a political campaign of a few years ago. It was first used in reference to some proposed system of municipal improvements which should make San Francisco the most beautiful city in America and a pleasure resort for the whole continent, but was later turned to the derogatory sense in which it appears in the paper. It died with the campaign, and it is not fair to use such casual expressions in a comparison of the solid merits and achievements of cities.

As to the 2 200 saloons in San Francisco, the Police Commission of San Francisco gives the number of retail liquor licenses now in force as 2 051, which includes some hundreds of restaurants where liquor is sold with meals.

The civic spirit of San Francisco is manifested in the attitude of the people toward public improvements. When shown the necessity or advisability of such works, they have been most liberal in voting bonds, although showing great intelligence and discrimination in demanding proof of the advantage to come from proposed expenditures. A great amount of money has been spent during the past ten years, and more is now being spent, on improvements the cost of which is approximately as follows:

New sewer system, nearly complete	$6 150 000
High-pressure fire system, acknowledged by all experts to be the best in the world, containing 72 miles of mains, covering all the principal business and residence districts, and reducing the insurance rates annually $1 250 000	5 750 000
Municipal street railways, 25 miles of double track, with rolling stock and car barns	5 500 000
Carried forward	$17 400 000

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Brought forward	$17 400 000
Asphalt plant	60 000
Civic center	8 800 000
Schools	7 500 000
Hospitals	3 000 000
Fire and police stations	510 000
Public library lands	740 000
Hall of Justice and County Jail	1 350 000
Hetch Hetchy water supply, lands and preliminary work	2 000 000
Docks	9 000 000
Streets and pavements	15 000 000
New parks and public playgrounds	1 360 000
Traffic tunnels	4 600 000
Total	$71 320 000

In addition to these municipal enterprises, about $11 000 000 has been expended on the Panama-Pacific International Exposition, the total expenditures on which will amount to about $18 000 000. Of this amount $7 500 000 was subscribed by corporations and individual citizens of San Francisco, and $5 000 000 was raised by the issue of municipal bonds.

The total amount of municipal bonds sold during the past ten years is $47 701 200. The City's bonds have generally commanded premiums, so that the total amount of money realized was about $49 253 000. The total amount authorized by the voters, but not yet sold, is $45 832 800, which includes $43 428 000 for the Hetch Hetchy water supply. The harbor bonds are a State issue, to which the citizens of the State contribute nothing, as the interest and sinking-fund payments come from the port revenues. The difference between the total cost of improvements mentioned and the total amount of bonds sold is accounted for by premiums on bonds and by payments for work out of current revenues and by assessment districts.

The municipal street railways constitute the first essay of San Francisco in the municipal ownership of public utilities, and the results are very gratifying. The first line was opened to traffic on December 28th, 1912, and at present there are 18 miles of double-track railways in operation, 4.5 miles under construction, and 2.5 miles being planned. The gross receipts during August, 1914, averaged $3 186 per day, and the operating expenses, $1 597 per day. The amount invested in the roads now in operation is about $4 000 000, on which the interest charge is about $525 per day. Liability insurance costs about $45 per day. Depreciation is estimated at 18% of the gross receipts, or $573 per day. The total daily expenses of the roads thus amount to $2 740, leaving a balance of $446 per day,

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which is a rate of more than $160 000 per year, or 4% of the investment—much more than the amount of the taxes that a private corporation would pay—available to be applied to extensions of the lines. This augurs well for the City's ability to manage successfully its own water system and the accompanying power development. Incidentally, it may be mentioned that San Francisco is the first city in America to achieve financial success in the operation of a municipally owned street railway system.

The writer apologizes for having taken up so much space with matter entirely irrelevant to the subject in hand, his excuse being that he wishes to correct any misapprehensions which might become fixed in the minds of the readers of the paper, if the statements contained therein were permitted to go uncorrected.

As to the water supply situation in the Bay cities, Mr. Cory presents considerable discussion on the present and possible future sources for San Francisco, beginning with the Spring Valley Water Company's system. For years there has been much investigation as to both the ultimate and the available yield of the Spring Valley sources. Many reports and estimates have been filed by various parties—some representing San Francisco or the East Bay cities, or both; others working in the interests of one or more of the water companies supplying those cities. In matters of this sort it usually happens that investigators of the latter class are very optimistic in their estimates, and those of the former class are more conservative. Table 1 shows the estimated yield of the various Spring Valley sources. The grand total yield of the Water Company's Peninsular and Alameda systems is estimated by Messrs. Herrmann, Schussler, Chittenden, and Powell, at about 220 000 000 gal. per day. These engineers were all engaged on the Spring Valley side of the controversy before the Secretary of the Interior regarding the City's proposed Hetch Hetchy development. Their estimates were reduced 50% by the U. S. Army Engineers, and, furthermore, it was shown that the yield was not obtainable, except at prohibitive cost, for municipal purposes. Mr. Dockweiler, representing the City in the same case, in his American-Cosumnes report, gave an estimate, also shown in Table 1, of 109 500 000 gal. per day—about half the amount of the estimates of the engineers on the other side. Later investigations prove even this to be excessive.

The run-off and discharge records mentioned by Mr. Cory on page 32 are not by any means all that could be expected of run-off and discharge records. Those on Calaveras Creek consist mainly of a series of records of depth and width and crude float observations over a very short course, taken by a watchman. No serious attempt at scientific stream gauging has been made until very recently. Members of the staff of the City Engineer of San Francisco, with the co-operation of the Spring Valley Water Company, are now making such

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measurements of stream flow from the Company's sources as will make possible more precise estimates of the possible water development from those sources than can be made at present from the rough and ready measurements of the past.

The same degree of uncertainty which exists with regard to the Spring Valley sources, and the same wide variation of estimates of possible yield, are encountered in the study of the sources from which the transbay cities draw their water supply. The situation in Alameda County is complicated further by the fact that the various streams, the water of which it is proposed to take for domestic supply, are the same streams that replenish the underground water supply of the valleys below, and any further withdrawal of water, beyond the quantities now taken, will be bitterly opposed by the farmers who use the underground supply for irrigation.

Yet, Mr. Cory, unmindful of the differences between engineers who have made long and careful study of the various sources, and not heeding the possible legal complications, states positively on page 36 that the total ultimate safe draft for municipal purposes for San Francisco and the Peninsula, the East Bay region, and Marin County and the North Bay region is 231 000 000 to 256 000 000 gal. per day, allowing a variation from minimum to maximum of only 25 000 000 gal.

It is certainly absurd for any one to assume that the engineers for the Bay cities have not given thorough investigation and consideration to the possibilities of the local sources of supply. In doing so they have analyzed all phases of the question, and have given due weight to inaccuracy of records and possible legal complications, and also to the fact that the development of some portions of the local sources would be so costly as to be prohibitive, unless there was absolutely no other available source.

For example, the coast streams of the Spring Valley Water Company were credited by Mr. Dockweiler, in his American-Cosumnes report, with a yield of 31 700 000 gal. per day, but to develop this water would be so expensive that ordinary financial prudence would demand that the money be used toward the development of the large supply, at less cost, which it is ultimately proposed to bring from the Sierra Nevada.

It is clear, to those who have investigated the situation from a fair and impartial standpoint, that the supply which can be developed economically from the local sources will be scarcely more than sufficient to keep pace with the normal growth of the cities and the suburban communities tributary thereto until the mountain water can be introduced.

Whether the problem is one for San Francisco or for "Greater San Francisco-Oakland", the answer is that San Francisco and Oakland

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are both "in the same boat" as regards water supply; both need immediate extensions, both are suffering from the exploitation of private speculative corporations, and San Francisco would welcome the co-operation of Oakland in the development of her water supply; but, if Oakland does not decide to assist, San Francisco must, of necessity, go ahead by herself.

Under the discussion of whether "Greater San Francisco-Oakland" can afford a Sierra supply, Mr. Cory, on page 39, asks: "Can San Francisco afford the luxury of a Sierra water supply at an immediate expense of more than 10% * * * of its present assessed valuation?" (The present assessed valuation is $646 700 065.) Also, on page 78, he states: "The City has revised the estimated cost of the first 160 000 000 gal. per day installation from Mr. Freeman's figure of $37 501 400 to $64 000 000, and probably the figures on the other installations as well."

The writer is at a loss to understand these statements. Mr. Freeman's estimate of the total cost of the initial installation, not including the cost of a city distributing system and the permanent power development, is given on page 160o of his published report of July, 1912:*

> "Total cost of supply, varying from maximum of 240 million gallons daily to minimum of 160 million gallons daily in extreme drought, delivered to Irvington gate house overlooking San Francisco Bay, with 50 millions delivered into San Francisco 3 miles from City Hall, and 50 millions into Crystal Springs Reservoir when desired.... $36 981 000
> "And also with 60 million gallons daily ready for Oakland, San Jose, etc."

Mr. Freeman's estimate of $36 981 000 (not $37 501 400), on the basis given by him, still stands. Thus far, no radical departure from the Freeman plans is proposed, except that it may be considered advisable to install initially a smaller pipe line, 45 miles long, across the San Joaquin Valley for a supply of 60 000 000 gal. per day, and to develop at once 50 000 hydro-electric horse-power. The cost developed by Mr. Freeman was based on the assumption that common labor will be paid a rate of $2.25 per day of 8 hours. The Charter of San Francisco requires that common labor be paid $3 per day on municipal work. It is still an open question whether this rate will be held to apply to all work outside of the City as well as that within the City limits. If it should be so, the effect will be to increase the cost of the whole project perhaps 20%, bringing the cost of the system, if built according to the Freeman plans, to about $45 000 000. (These figures do not include a city distributing system.)

* "The Hetch Hetchy Water Supply for San Francisco, 1912: Report."

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On page 78 the statement occurs: "There are about 100 miles of steel pipe, some of it under heads as high as 600 to 900 ft." This is erroneous, as the Hetch Hetchy aqueduct, in the Freeman plans, from the Early Intake to San Francisco, contains 66 miles of tunnel and 87 miles of pipe. The highest head (except in the penstocks of the power-houses) is near Oakdale, at the east end of the San Joaquin Valley crossing, and is considerably less than 600 ft. Mr. Cory is in error here.

In calculating on other sources of water supply, the same prices of labor and material were used as in the estimate of cost of the Hetch Hetchy scheme, with the exception of the estimate for the filtered supply from the Sacramento River, in which Mr. Hazen used $3 per day of 8 hours as the wages of common labor. Thus, the question of unit prices does not enter into the comparison of various sources.

This matter was thoroughly understood by the Board of U. S. Army Engineers which acted in an advisory capacity to the Secretary of the Interior regarding the grant of the reservoir sites and aqueduct rights of way. The members of this Board were selected as competent and impartial judges, and not only compared the estimates submitted to them, but made their own independent estimates on the same basis. These covered nearly all the possible sources mentioned by Mr. Cory on pages 62 to 76, including Coast Range sources, supplies from the delta of the Sacramento and San Joaquin Rivers, and Sierra sources from the McCloud River to the Tuolumne. They heard or read all the testimony submitted by the City of San Francisco, the Spring Valley Water Company, and promoters interested in various possible competing sources of supply, and confirmed the judgment of the City's engineers that, considering the problem from the standpoints of both engineering and economics, the Hetch Hetchy is the most desirable. To quote their own words, from page 50:*

"The project proposed by the City of San Francisco, known as the Hetch Hetchy project, is about $20 000 000 cheaper than any other feasible project for furnishing an adequate supply."

Relative to this matter, the following extract, from page 14 of that report, is of interest:

"In one important respect the situation in California requires special consideration. * * * In California all water has great value. Due to the large extent of arid and semiarid land that can be made fertile by the use of water, irrigation is assuming great importance; due to lack of coal and the opportunity for economical water-power development the use for the latter purpose will surely be greatly extended. In a relatively few years practically all available water will doubtless be appropriated for one or the other purpose,

* "Hetch Hetchy Valley: Report of Advisory Board of Army Engineers to the Secretary of the Interior," February 19th, 1913.

and it will then be possible to obtain it for municipal use only at great cost and damage to existing communities and industries. It is, therefore, necessary to-day for the cities of California to look further ahead than in most other parts of the country, and to take such steps that in the future when they may need the water they shall have the right to it. For this reason it is believed that in making provision for the future supply of San Francisco and other bay cities a source should be selected if possible that is capable of supplying the needs of the communities for the balance of this century. Such a course would seem both wise and reasonable, provided it involves no sacrifice of economy." Mr. O'Shaughnessy.

This answers the question as to whether or not the Bay cities can afford a Sierra supply. Their material, social, and commercial development depends on their obtaining additional water; the Hetch Hetchy has been shown to be the most economical and adequate source; and between the alternatives of providing additional pure water supply by the community or civic stagnation under existing conditions, there can be but one answer to the question.

The Raker Bill, which grants to San Francisco the necessary reservoir sites and aqueduct rights of way through the public lands, was a compromise between the conditions on which the City first sought the necessary rights for the development of the Hetch Hetchy water supply, and those which the irrigationists sought to impose. In framing the bill due regard was also given to the consideration that the National Park should not be treated in any way that would detract from its status as a pleasure ground for the public. A great deal of thought was devoted to the formulation of this bill by the City's engineers and attorneys, who did not wish at any time to take from the irrigationists any water that they might usefully apply to their lands and be legally entitled to. The City has always realized that its prosperity and that of its back country are to a large extent mutually dependent, and, further, does not wish to antagonize the irrigation districts by taking from them something which they feel belongs to them. The demands of the irrigationists have been increased several times since the City made its initial application to the Secretary of the Interior, and the final result was that the City consented to the practical guaranty that the irrigationists should have water to the extent now legally diverted by them in their two canals, Turlock and Modesto, namely, the full natural flow of the Tuolumne and its tributaries up to 2 350 sec-ft. at La Grange diverting dam when the districts can beneficially use it, and 4 000 sec-ft. from April 15th to June 14th of each year.

This arrangement, however, was only consented to after the City's engineers had thoroughly investigated the available water supply of the Tuolumne, revised their former figures, and found that, even with

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the restrictions imposed, there would still be sufficient water to fulfill the conditions and leave the City's own future supply up to 400 000 000 gal. daily for domestic purposes unimpaired. In confirmation of this, the following finding of the Army Board is conclusive:

"The board is of the opinion that the use of the Hetch Hetchy Valley as a reservoir site is necessary if the full flow of the upper Tuolumne is to be conserved.

"The board further believes that there will be sufficient water if adequately stored and economically used to supply both the reasonable demand of the bay communities and the reasonable needs of the Turlock-Modesto Irrigation District for the remainder of this century."

The Raker Bill, as passed, has been reviewed by Mr. Freeman, who was one of the principal consulting engineers for the City. On March 21st, 1914, he submitted a report to the City Attorney and the City Engineer entitled, "Reservoirs Ultimately Required for a Hetch Hetchy Water Supply of 400 Million Gallons Daily for San Francisco under the Terms of the Raker Bill". The following is quoted from that report:

"I submit herewith my report upon the additional burden imposed upon the City of San Francisco by increased water rights given to the irrigation districts by the terms of the recent grant by Congress to the City of San Francisco, of rights of way and reservoir sites in the Hetch Hetchy region, by House Bill No. 7207, known as 'The Raker Bill'. I am pleased to report that a most painstaking study proves that the City can never suffer seriously from this cause, and the conclusion is justified anew, that it was ten-fold better for the City to make these concessions than to imperil the passage of the Hetch Hetchy bill, and that the total storage of 190 billion gallons, or say in round numbers, 200 billion gallons, found necessary for a dependable water supply of 400 million gallons daily in a series of years of lowest rainfall ever yet known, by my new computations, can be had in the City's three reservoir sites, Hetchy, Eleanor, and Cherry, with the addition of Vernon Lake and Poopenaut.

"Altho the terms of the grant are now fixed beyond change, it has appeared wise to check up the hurried computations made while the Bill was being negotiated before Congressional committees last Summer and review all data in the most painstaking way, particularly since the wisdom of certain conclusions reached by the City's representatives has been questioned by certain good citizens before one of your most important civic organizations, and also because there is now available the daily record of gagings at the City's dam sites at Hetchy and Eleanor thruout the remarkable severe drought of 1912-1913, to which the priorities specified in the Raker Bill can be directly applied, affording a more variable and convincing demonstration."

The additional burden on the City, due to the terms of the bill, is seen from this report to be merely a matter of constructing some

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comparatively small and inexpensive reservoirs in addition to the Hetch Hetchy, Eleanor, and Cherry Reservoirs, which have been contemplated all along.

There are reservoir sites available to San Francisco and to the irrigation districts, with capacities which total much more than the maximum storage which will ever be required.

The Raker Bill sets aside for the use of the City all reservoir sites above the mouth of Jawbone Creek (a tributary entering the main Tuolumne River about 15 miles below the Hetch Hetchy dam site), leaving the irrigationists the right to develop storage below that point. The following reservoir sites are thus available for San Francisco:

	Capacity, in acre-feet.
Hetch Hetchy Valley (300-ft. dam)	344 000
Lake Eleanor (250-ft. dam)	288 800
(Mr. Freeman figures on a 150-ft. dam giving 37 000 000 000 gallons or 113 500 acre-ft. capacity.)	
Cherry Valley (150-ft. dam)	56 800
Vernon Lake	47 900
Wilmer Lake	5 000
Tilden Lake	27 800
Emigrant Lake	14 300
Huckleberry Lake	52 200
Poopenaut Valley	44 300
Cherry Mouth (at junction of Tuolumne and Cherry Rivers)	8 200
Total	889 300

or 290 000 000 000 gal., which is 90 000 000 000 gal. more than is necessary, in Mr. Freeman's opinion, as previously quoted, for the City's development of 400 000 000 gal. per day.

Below Jawbone Creek, and contiguous to the Tuolumne water-shed, the following sites remain:

	Capacity, in acre-feet.
Empire Mills Reservoir site	35 000
Coffin Hollow	12 000
Lily Lake	15 000
Cottonwood	25 000
Ackerson	26 000
Stone	26 000
Fernbrook	26 000
Heardin	25 000
Carried forward	190 000

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	Capacity, in acre-feet.
Brought forward	190 000
Big Creek	12 500
Moccasin Creek	12 000
Upper La Grange	200 000
Dry Creek	325 000
Davis Reservoir (already constructed)	48 000
Dickinson Reservoir	60 000
Dallas and Warner Lakes (storage of 30 000 acre-ft. already developed)	60 000
Total	907 500

or 296 000 000 000 gal.

All these reservoir sites, with the possible exception of a few of the smallest, are capable of development at an economical cost per acre-foot stored.

Pending the construction of additional reservoirs by the irrigation districts, the Raker Bill provides that whenever the districts desire water in excess of that to which they are entitled under the terms of the bill, the City, on the demand of the districts, shall sell such quantities of stored water as may be needed for the beneficial use of the districts—without impairing the supply for domestic requirements—at such a price as will return to the City the actual total cost of providing such stored water, including a fair proportion of the cost to the City of the conduit, lands, dams, and water-supply system included in the Hetch Hetchy and Lake Eleanor sites. It is provided, however, that the City may require the irrigation districts to purchase a minimum quantity of stored water, and shall not be required to furnish more than a maximum quantity, in any calendar year, the minimum and maximum quantities to be fixed by the Secretary of the Interior, keeping in mind the superior needs of domestic use; and, further, the City is not to be required to furnish more than the minimum quantity until the districts shall have first drawn on their own stored water to the fullest practicable extent.

No hardship on the City is to be apprehended from the obligation under these terms to furnish water to the irrigationists. The latter have already developed storage amounting to 78 000 acre-ft., which is capable of expansion, as already shown, to a final total of more than 900 000 acre-ft. Their dams and appurtenances are not and need not be of the relatively expensive type required for the purposes of a domestic water supply, and, therefore, the provision that the water purchased by them shall be paid for at a price which will include a fair allowance for the cost of construction should certainly induce them to construct their own reservoirs.

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Furthermore, the irrigation districts have the option to purchase surplus electric power which the City may not need, for the use of any land owner or owners therein for pumping sub-surface water for drainage or irrigation at cost. This will make economically available to the irrigators the vast underground water supply which lies only a few feet beneath the ground surface but is as yet practically untouched, owing to the prohibitive cost of steam power in small units and electric power purchased from corporations whose prices include the profits of their stockholders.

The districts have already spent considerable money in efforts at drainage of sub-surface water due to the rising of the ground-water from over-irrigation, and a generous supply of power to aid them in this policy will far outweigh any sentimental objections to San Francisco's storage of waste flood-waters on the higher mountain levels.

It is not within the range of probability that if during any year the City's supply of stored water should be brought so low as to make a water famine imminent, she should still be compelled to furnish water to the districts, particularly if they had not used the same diligence in developing the reservoir sites available to them that would be expected of the City both under the terms of the Raker Bill and under the dictates of ordinary foresight and prudence.

Turning now to the conclusions on pages 85 to 87, the writer will endeavor to answer, from the standpoint of a citizen of San Francisco and, to the best of his ability, from the standpoint of a citizen of the proposed Metropolitan District as a whole, the questions that Mr. Cory propounds to the citizens of San Francisco.

The cities of the Metropolitan District should naturally develop together, and all the cities concerned are facing a shortage of water; their development depends on their securing an additional supply at once; their interests are interwoven—the problem of supply is certainly one for a metropolitan water district to handle.

As already stated, San Francisco has attempted to get the transbay cities interested with her in securing additional water supply. Due doubtless to the selfish interests at work, those cities have not yet responded to San Francisco's invitation. There is little doubt, however, that they will eventually do so. It happens, however, that the most available source of supply for the Metropolitan District is also the most available source for San Francisco alone, and as the demand for additional water is on us, San Francisco must proceed independently with her development, until such time as the transbay cities may see fit to enter into the work and share the burden with her.

The studies which have been carried on during the past few years have convinced the City's engineers that the economically available increase of the water supply from the Spring Valley sources is hardly more than sufficient to cover the present unsatisfied demand and the

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natural increase of consumption during the period of construction of the Hetch Hetchy works.

The Hetch Hetchy scheme has been the subject of very serious consideration by many engineers who, after comparing it with all other available sources of water supply, have endorsed it. The engineers who have questioned it, with the sole exception of those employed by the irrigation districts, have been the representatives of special competing interests, hoping to dispose of their own private holdings for their own private gains. The Raker Bill, as passed, imposes on the City a slight additional burden to that which it has always expected to sustain, in the form of obligations not to interfere with the developed water supply of the irrigation districts, and the City, by consenting to the conditions of the bill, has eliminated the unjust accusation that she was trying to gain something at the expense of the irrigationists.

The securing of the Hetch Hetchy water supply is not a matter of sentiment with the people of San Francisco in the limited sense in which the word "sentiment" is used by Mr. Cory. The sentiment of the people is for the development of their City. Their accredited representatives have shown, to their satisfaction, that the Hetch Hetchy is the best source of water supply available, and they are willing to invest in it, not "sums so vast that many other requirements more vital to the interest of the City must be sacrificed or skimped", but the reasonable and necessary cost to secure the water supply which is essential to the growth of the City and the expansion of its enterprises.

The acquisition of the present water systems of San Francisco and of the Bay cities, either by a Metropolitan District, or, pending the formation of such a district, by the cities acting independently, is a very desirable step toward the development of a well-balanced metropolitan system.

Mr. Grunsky,* in his discussion, states correctly that in Mr. Cory's paper there is a marked departure from the ordinary treatment by an engineer of an engineering problem. At and previous to the time the paper was prepared its author had never been officially employed on an investigation of the subject therein discussed. It is therefore natural that any engineering data presented should be merely a résumé of the studies of other investigators, and since no new light is thrown on the subject, the matter therein is of little importance to the Profession at large. Mr. Grunsky, however, as noted in his discussion, held the position of City Engineer of San Francisco from 1900 to 1904. As the former occupancy of that office lends weight

* The following part of Mr. O'Shaughnessy's discussion was written after reading the discussion by Mr. Grunsky.

to his statements on the water question, it is necessary that some of the erroneous impressions conveyed by his discussion be corrected. Mr. O'Shaughnessy.

Mr. Grunsky calls attention to the fact that the Hetch Hetchy project under the Raker Bill is not his Hetch Hetchy project. Despite Mr. Grunsky's best efforts, his application, filed in 1901 with the Secretary of the Interior, for a permit to impound water at Hetch Hetchy and Lake Eleanor and conduct 60 000 000 gal. daily to the City was denied. In 1905 the application was renewed, but permission was again refused.

In 1908 Secretary Garfield issued a revokable permit allowing the construction of the Eleanor Dam as a first unit of construction, but requiring the full development of the Lake Eleanor water-shed before any work could be done in the Hetch Hetchy Valley. Only 79 sq. miles of water-shed are tributary to the Eleanor Dam site, on which a dam, 1 700 ft. long on the crest, would have to be constructed to impound a run-off of only 20 billion gallons in ordinary seasons, and in years of minimum precipitation 8.7 billion gallons. In contrast, the area tributary to the Hetch Hetchy dam site is 459 sq. miles, or 5½ times that of Lake Eleanor, and a dam 300 ft. high and 700 ft. long will impound 112 billion gallons. The Eleanor Dam site is much more inaccessible than the Hetch Hetchy, and unit prices for construction materials delivered would be correspondingly greater. Despite the fact that the Garfield permit would require a vastly greater initial expenditure to develop far less water, and after the City had voted $45 000 000 for constructing the Hetch Hetchy-Lake Eleanor system, in February, 1910, Secretary Ballinger called on San Francisco to show cause why the Hetch Hetchy Valley and reservoir site should not be permanently denied San Francisco and eliminated from the above permit. After fighting for nine years, the City had nothing tangible to show for the money it had expended. Then, to quote Mr. Grunsky:

"San Francisco was thus put on the defensive and, due to lack of confidence of various of her official departments in each other, placed this case in the hands of an expert called in from the East."

This expert, of whose employment Mr. Grunsky declares with some bitterness "the natural result was that thousands of dollars were needlessly expended in the accumulation of a mass of statistical information", was John R. Freeman, M. Am. Soc. C. E., whose masterly presentation of accurate statistical information on this project to the Board of Army Engineers was one of the most potent factors in securing for San Francisco the Hetch Hetchy grant. The subsequent report rendered to the Secretary of the Interior by the Board of Army Engineers substantiated practically every claim made by Mr. Freeman. Mr. Freeman's report, augmented by the oral testi-

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mony of the City Engineer of San Francisco, Percy V. Long, City Attorney, and Colonels Biddle, Cosby, and Taylor, who constituted the Advisory Board of Army Engineers, resulted in the enactment by Congress of the Raker Law.

Mr. Grunsky speaks of extraordinary expenditures and lack of study. He neglects to mention that if there were any extraordinary expenditures he was a beneficiary thereof as he received compensation for studies of the Mokelumne River, the McCloud project, and for other services. Possibly Mr. Grunsky's professional pride is hurt because his report on the Mokelumne River was not presented by the City when the case was pending before the Secretary of the Interior. The report was so hastily and disjointedly prepared, and contained such insufficient data that Mr. Freeman and the Army Engineers gave it no consideration. This, together with the fact that Mr. Grunsky's original plans were radically changed, undoubtedly inspires much of his criticism.

Under the provisions of the Raker Act, San Francisco is given the right to store water in the Hetch Hetchy Valley, Lake Eleanor, and Cherry Creek Reservoirs without any restrictions as to the order of development, as well as to construct and maintain power-houses, transmission lines, conduits, pressure tunnels, telephone and telegraph lines, and all appurtenances necessary for the operation of the municipal water-works and power supply. San Francisco must recognize the prior rights of the Modesto and Turlock Irrigation Districts to receive 2 350 sec-ft. when the same can be used beneficially to irrigate their lands and when that quantity of water is flowing in the Tuolumne River, provided said lands shall not exceed 300 000 acres; and San Francisco must further permit 3 000 sec-ft. of the natural daily river flow during the 60-day period immediately following and including April 15th of each year to pass its impounding dams, should this quantity ever be essential to the needs of the Irrigation Districts. Although the Raker Bill limits the irrigation obligations to 300 000 acres, the United States Army Board found that, with proper storage, there was ample for 400 000 000 gal. daily for San Francisco and for 400 000 acres of land.

In the Hetch Hetchy Valley, Lake Eleanor, and Cherry Creek Reservoirs, San Francisco can store economically 689 700 acre-ft. By utilizing other available storage sites within its drainage area, a total of 836 800 acre-ft. can be made available. According to the findings of the Board of Army Engineers* "Four hundred M. G. D. may be supplied San Francisco from the high areas without interfering with Turlock and Modesto rights, with 560 000 acre-feet of storage". The difference between 689 700 and 560 000 is the substan-

* "Hetch Hetchy Valley: Report of Advisory Board of Army Engineers to the Secretary of the Interior, February 19th, 1913," p. 110.

tial excess of 129 700 acre-ft. which San Francisco would have as surplus storage after supplying all needs, by developing Hetch Hetchy, Eleanor, and Cherry. Mr. O'Shaughnessy.

In reservoirs outside the area controlled by San Francisco, the Turlock and Modesto Irrigation Districts can conserve 960 000 acre-ft.; 168 000 from its foot-hill reservoirs and 792 000 from the Upper Tuolumne. According to the report of the Army Engineers, previously referred to:

"The irrigation requirements of the Turlock and Modesto irrigation districts may be provided by their existing rights if conserved by means of 370 000 acre-feet of storage capacity."

From these figures it is evident that in the Tuolumne River drainage area there is ample water to supply the demands of both San Francisco and the Irrigation Districts. Mr. Grunsky's doubts on this subject are no more scientifically justifiable than his assumptions on the invisible leakage from Crystal Springs Reservoir. Even were the danger of a shortage remotely possible, it must be remembered that, before exercising their priority to its limit, the irrigationists are required to develop their own local resources, and that it is expressly stipulated in the Act that the laws of the State of California relating to water shall in any event control as to the priority needs and use of water. By these State laws the City's interests are amply protected. Had no concessions been granted to the Irrigation Districts, San Francisco would not now enjoy a Federal Grant by which it is assured a permanent supply of 400 million gallons of pure mountain water daily. Moreover, as explained by the Army Board to the Committee on Public Lands, should a controversy ever arise between the City and the irrigationists as to their respective rights, domestic consumption, being the highest use to which water can be applied, under the laws of the State of California, the rights of the City will be paramount.

Mr. Grunsky regrets that:

"San Francisco, under the Raker Law, is to expend from $500 000 to $1 000 000 on the proposed storage works in excess of her own requirements, in order to be in a better position to meet the obligation, which she has apparently assumed, to supply some stored water at cost to the irrigation districts.

"San Francisco, [he says] under the same law, will be required to pay for the appropriated water, which should be hers without cost, under State law, at the rate of $15 000 for 10 years beginning in 1919, thereafter $20 000 per year for 10 years, and thereafter $30 000 per year."

Very few will agree with Mr. Grunsky that to make a capital outlay of from $500 000 to $1 000 000 for an increase of water supply from 60 million gallons to 400 million gallons daily is onerous, when

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the cost of developing the 60 million gallons daily would be approximately $45 000 000. The money paid by the City to the Government after 1919 is, according to the provisions of the Raker Bill "to be applied to the building and maintenance of roads and trails and other improvements in the Yosemite National Park, and other national parks in the State of California". San Francisco, being the commercial metropolis of California and the clearing house of the Pacific Coast, will naturally profit more by the expenditure of this money than any other city in the State. The prosperity of the Irrigation Districts and the enhancement of the attractiveness of the State's parks, which are visited yearly by thousands of Eastern tourists who make San Francisco their final destination, are of prime importance to the City, and money spent for these purposes will not be wasted. Certainly, it is not to be caviled at, in view of the enormous power concessions granted to San Francisco under the Raker Act.

In Mr. Grunsky's project power was to be developed sufficient merely to pump 60 000 000 gal. daily into San Francisco. The value of the power rights acquired with the enactment of the Raker Bill is conservatively estimated at $45 000 000, on the basis of the findings of the Army Board that 115 000 h.p. could be developed economically. Subsequent studies have disclosed the fact that 200 000 h.p. can be made available from San Francisco's Sierra sources. The percentage of this which will be used by the Irrigation Districts at cost is small. Mr. Grunsky believes that with oil at 75 cents per bbl. power can probably be developed more cheaply by steam in San Francisco than generated by water in the Sierras and transmitted a distance of 150 miles. The refutation of this theory is a matter of record in Case No. 400 before the Railroad Commission of California. In this case, in which the Pacific Gas and Electric Company was under investigation, both hydro-electric and steam plants were studied in order to determine a proper cost for electricity delivered at sub-stations for distribution. There are ten hydro-electric plants, and they include stations with capacities from 660 to 40 000 kw., having a combined capacity of 107 310 kw. There are four steam plants in the Pacific Gas and Electric system, totaling 70 200 kw. in output. Two of these plants have a combined capacity of 63 000 kw. in Curtis turbines. The capital cost of the hydro-electric plants is approximately $136.50 per kw., and the cost of the steam plants is $77.70. The average cost of energy deliverable at sub-stations from hydro-electric plants was $0.00488 per kw-hour. The average energy cost deliverable to transmission lines from steam plants was $0.01248 per kw-hour. These two costs include interest at 8%, depreciation, maintenance, and all operating expenses of generation. The cost of oil at the Oakland and San Francisco plants for the year considered was $0.68½ per bbl. To the cost for hydro-electric current should be added the cost for

transmission and delivery to the sub-station. This amounted to $0.00177 per kw-hour. This brings the cost of hydro-electric current up to $0.00665 per kw-hour. These figures were made in July, 1914, for the purpose of fixing rates for the Town of Antioch. The State Railroad Commission has always resolved the doubt in favor of the corporation, when compiling data for the purpose of fixing rates, and has, no doubt, also allowed the Pacific Gas and Electric Company a considerable sum for capital expenditures made on property which has not as yet been developed to its full capacity. The developments considered in this report are all properties which were constructed primarily for the development of power, and are not plants from which power is merely a by-product, as in the Hetch Hetchy project, where the City must store water for domestic use, and can generate power with a very small additional expenditure. Mr. Grunsky's statement implies that all the electric companies which so recently have expended millions on hydro-electric development in the Sierras, have been guilty of gross misjudgment. If, as Mr. Grunsky states, the power feature is likely to prove a burden, then "the yoke is light and the burden sweet." Mr. O'Shaughnessy.

For an engineer to take issue on a point of law with attorneys and bonding experts of national reputation, who have passed on the validity of the $45 000 000 bond issue to construct the project approximately according to the Freeman plan, seems to be ill advised. It is likewise inconceivable that the people of San Francisco should object to this expenditure, to secure 400 million gallons daily, when they approved the bonds in the expectation of receiving only 60 million gallons daily.

Mr. Grunsky's innuendo, that the City Engineer has "a hand in the game", insinuating that politics is a governing feature in the conduct of the Bureau of Engineering is unjust, as is his statement that the natural results of Mr. Freeman's employment by San Francisco "was that thousands of dollars were needlessly expended". Every unprejudiced citizen of San Francisco, including many of those journalists who decried municipal ownership most energetically and fought hardest against the voting of bonds for the municipal system, agrees that the planning and construction of San Francisco's Municipal Railroads by the present City Engineer were accomplished with good judgment and efficiency. The phenomenal success of this enterprise would have been impossible had the City Engineer, to whom this success is in large measure due, been hampered by politics. The same applies to all of the City's engineering construction that has been undertaken within the past few years.

To prove his contention Mr. Grunsky states:

"Within a year there was named a Board of Consulting Engineers to pass upon the hydro-electric features of the first power installation

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under the unauthorized project, understood to be 50 000 h.p. These engineers were designated by name in the City Engineer's request for their appointment. From this circumstance alone it is to be inferred that the political management of the city's water project will be continued."

By what intricate principles of logic Mr. Grunsky derives the conclusion from the premises is not apparent. When it becomes necessary for the chief engineer of a private corporation to request the directors for an appropriation for the temporary employment of one or more consulting engineers, he invariably names the engineers he desires to collaborate with him. Moreover, the personnel of the Board of Consulting Engineers precludes the idea of political preferment. It consisted of F. G. Baum, M. Am. Soc. C. E., retained as consulting engineer on almost every large hydro-electric development on the Pacific Coast and in South America; W. F. Durand, Professor of Mechanical Engineering at Stanford University and Consulting Engineer for Los Angeles power projects; and John D. Galloway, M. Am. Soc. C. E., who has had extensive experience in high-head power plants.

Continuing, Mr. Grunsky discusses the proposed purchase by the City of the properties of the Spring Valley Water Company. He states:

"According to the latest proposition, to be voted on some time next spring, the Water Company will be allowed to retain several thousand acres of valuable land in and adjoining San Francisco, besides some 5 000 acres of land in Livermore Valley. The value of the Pleasanton wells as a part of the system will be reduced because the maximum draft on them is to be limited to 15 000 000 gal. per day, and because the ownership of the 5 000 acres will carry with it the right to draw on the Livermore gravels for all the water that may be needed on these lands * * * there may not be much water left in the gravels for San Francisco."

The City Engineer of San Francisco was directed by the Board of Supervisors, in February, 1913, to designate such lands and properties of the Spring Valley Water Company as he considered it advisable for the City to acquire for use in connection with the early completion of the Hetch Hetchy project. After a very careful and comprehensive study he decided that of the lands owned by the Company at that time, amounting to more than 100 000 acres, it would be advisable for the City to acquire about 68 000 acres. The remaining lands comprised properties which are not desirable for use in connection with either the Spring Valley or the Hetch Hetchy system, some because the development of water from the sources in question would be uneconomical, some because the quality of the water is not satisfactory, and some because the right to divert water to the City would entail the acquisition of expensive riparian rights. In

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regard to the 5 000 acres of land in Livermore Valley excluded from the proposed purchase, the City Engineer only consented to the exclusion of these lands with the understanding that the City should have the right to take water from this region to the amount of, but not in excess of, 15 million gallons during any one day, this right being without hindrance or conditions of any sort, the Spring Valley Water Company waiving any and all rights to object to or enjoin the taking of such water, either on the ground that the pumping of underground water to the limit of 15 million gallons daily lowers the subterranean water plane under the lands retained by the Spring Valley Water Company, or that it drains the water beneath the retained lands, or on any other ground whatsoever. As 15 million gallons per day is the ultimate yield to which the Livermore gravels can be economically developed, the restriction of which Mr. Grunsky complains is not an onerous condition.

Mr. Grunsky states that the Hetch Hetchy project has not "yet received the necessary study to determine its main structural features." Enough study has been given the subject to determine definitely that the Grunsky scheme, involving many miles of open ditches, should be abandoned, and active construction has been started on the pressure system.

Mr. Grunsky's complaint of the compensatory features of the Bill was expressed by representatives of hydro-electric power companies in Washington.

There was written into the Hetch Hetchy Bill the conservation principles of the Roosevelt, Taft, and Wilson administrations, and the Federal compensation and regulation are phases of those conservation policies.

These questions were fully debated in the House and Senate, and in the House the vote was more than four to one for the Bill; in the Senate, two to one. Therefore, criticisms of these features are quarrels with the Federal Government policies, which, by the way, the City of San Francisco, so far as the Hetch Hetchy project is concerned, has approved unanimously.

Mr. Hazen.

ALLEN HAZEN,* M. AM. SOC. C. E. (by letter).—The water supply conditions in San Francisco are unique in some respects. Notwithstanding the exhaustive treatment of the author, some of the peculiarities of the situation do not seem to be adequately reflected in his paper. The comparison between San Francisco and Los Angeles is a natural one; it is one that has been made very frequently, and will continue to be made, although there are fundamental differences in the water supply conditions of the two cities.

Los Angeles was situated on a water supply. The primary reason for its existence was that water was readily obtained from the Los

* New York City.

Mr. Hazen.

Angeles River. This excellent supply, carefully developed, has served the population of the Los Angeles district up to the present time, and it is capable of a certain further development. This water is close at hand and at a relatively high elevation, so that it can be, and is, delivered in the city at a very low unit cost. Under these conditions, low water rates have been possible and are natural.

The people of Los Angeles, realizing the limit of the Los Angeles River supply and the effect of that limit on the ultimate growth of the city, went about securing a new supply from the Sierras. The undertaking has been carried out during the last 10 years in a way that commands the admiration of the world. In building these works, the city was extraordinarily fortunate in finding a route for an aqueduct on a continuous down grade from the source in the mountains to the city. Both the source and the intervening land were at sufficient elevations to allow a gravity supply. Indeed, the elevations were more than sufficient, and a large surplus head is available for use in developing electric power. Under these conditions, notwithstanding the long distance, the cost of introducing Sierra water has been surprisingly low.

San Francisco is situated on a harbor. The harbor is a magnificent one, and justifies the location. The site of the city was no doubt selected without regard to water supply, and local sources have proved sufficient for many years, although, as the years have gone by, it has been necessary to go farther and farther afield and to build new works at increasing cost. As time goes on it will be necessary to go still farther, and finally to introduce water from sources entirely outside the district.

A Sierra supply can be brought to San Francisco, but, unlike Los Angeles, an economical aqueduct location from the Sierras is not possible. A mountain range must be crossed, and this lies between two wide valleys but little above sea level. Under these conditions, although the distance is less, the cost of a suitable aqueduct will be increased many fold.

Sources now in use are sufficient to supply San Francisco and the whole surrounding district for some years, although additional storage, aqueducts, and other works are urgently needed to make these sources fully available for present and future needs. By going farther there are many possibilities of additional supply, and ultimately the quantity of water that can be made available, if needed for San Francisco, is enormously greater than the quantity that can be made available for Los Angeles; but the cost to San Francisco of the great supplies of the future will be far greater than that of the supplies which Los Angeles has utilized.

The cost of water in San Francisco and in the San Francisco district will never be as low as it is in Los Angeles. The reason is

found in the great natural difficulties, and in the cost of the works required to gather, transport, and distribute water in the San Francisco district. Mr. Hazen.

The people of San Francisco may regret that they cannot have water at rates that are current in Los Angeles, but they must remember that the higher rates are part of the price that they must pay for their location on their unique harbor. The harbor is worth the price. If water supply instead of harbor facilities had been the dominating influence in locating San Francisco, it would have been built at Sacramento. There water of excellent quality could have been supplied at a price, except for the cost of pumping, as low as water is supplied in Los Angeles or in any city in the country.

J. D. GALLOWAY,* M. AM. SOC. C. E. (by letter).—As a part of his discussion of Mr. Cory's paper, Mr. Grunsky states the following: Mr. Galloway.

"At the time the city thus voted bonds, there should have been created a Board of Water Supply, free from politics. This was not done. The administration at that time and the administrations that have followed, would not let go. The Mayor, who names the Board of Public Works, and who practically selects the City Engineer for the Board of Public Works, the Supervisors, with the Public Service and Finance Committees, the City Attorney, the Board of Public Works, and the City Engineer, all have a hand in the game.

"There is no programme as yet for any change. Within a year there was named a Board of Consulting Engineers to pass upon the hydro-electric features of the first power installation under the unauthorized project, understood to be for 50 000 h.p. These engineers were designated by name in the City Engineer's request for their appointment. From this circumstance alone, it is to be inferred that the political management of the city's water project will be continued. Deplorable as this may seem, it is yet possible that competent management will not be too greatly hampered by political obstruction and political direction; and the hope may be expressed that, despite the unfortunate circumstances of the past, San Francisco may yet achieve an adequate and in every way satisfactory water supply."

As one of the Board of Consulting Engineers on the power development, mentioned by Mr. Grunsky, the writer desires to make the following explanation, in order that the apparent strictures on the Board by Mr. Grunsky may be understood.

The definite plan of a water supply, on which was based the bill granting the right of San Francisco to use the Hetch Hetchy Valley as a storage reservoir, was made by John R. Freeman, M. Am. Soc. C. E., in a very comprehensive report on the entire situation. A clause in the bill required that within a certain definite time the City should file with the proper department at Washington maps showing the definite location of the conduits to be used, so far as they crossed

* San Francisco, Cal.

Mr. Galloway.

Government lands. Mr. Freeman's report was very complete as a whole, but, necessarily, the details, in part, required further consideration. This was particularly so in the case of the development of power as an adjunct of the water supply. Mr. Freeman's suggested design had in view the use of a tunnel for the conduit, leading to a point above one of the proposed power-houses, and thence a steep pressure tunnel in the rock down to the power-house, under a head of about 1 300 ft. As a means of controlling the possible surges in the water conduits, Mr. Freeman had suggested a surge chamber at the junction of the conduit tunnel and the pressure tunnel to the power-house.

It is a well-known fact that though surge chambers of proper dimensions and shape will provide for proper speed regulation of hydro-electric units, they do not in any way provide for taking care of the daily variations in power demand, designated as load factor. The flow of the water in the conduit being practically uniform, the maximum demand on the power-station is limited to the power which can be supplied by the maximum flow of the conduit. If, on the other hand, it is possible to supply at the lower end of the conduit, at a point just above the power-station, a reservoir of sufficient size, the daily fluctuations in power demand can be supplied therefrom, the uniform flow of the conduit passing at all times into the regulating reservoir. It is a rare thing for any power system to have a load factor of much more than 50%, and a number of the important systems of the country have load factors of less than this. In Mr. Freeman's report, the power development was considered as incidental to the water supply. The conduit was considered primarily as one which was to bring water to San Francisco, and the peak load of the power-plant was limited to the maximum capacity of the aqueduct tunnel, more than 19 miles long.

In requesting the Board of Consulting Engineers to serve, Mr. O'Shaughnessy stated that he was confronted with the necessity of presenting maps showing the final location of the conduits, within a limited time; and he also stated that a further study should be made of the power-station design, in order to see if the power development could not be made so as to obtain the maximum development of power, having in view a possible load factor, and without interfering at all with the flow of the water. To answer the various questions asked by the City Engineer really required, on the part of the Board of Consulting Engineers, a complete study of the entire situation of the development of power at the lower plant of Mr. Freeman's design. It is not necessary to dwell on the results of the study as made.

The writer wishes to make clear this point, that the City Engineer was under the necessity of determining these several features before a given definite date. Some appreciation of the problem can be gained when it is said that on a load factor of 50%, the station would

Mr. Galloway.

generate a peak load of about 100 000 kw. The Board of Consulting Engineers could not answer the questions without a complete study of the situation, and the City Engineer was hardly justified in determining the final location of the project until it had been examined.

To one who reads the statements made by Mr. Grunsky, it may appear that politics formed the principal element in the appointment of the Board of Consulting Engineers. In order that the readers of this discussion may form their own opinions, the following information is given: The Board was formed of Professor W. F. Durand, head of the Department of Mechanical Engineering at Stanford University, Mr. F. G. Baum, and the writer, both Members of this Society, in independent practice in San Francisco. Professor Durand, in addition to his duties at the University, has had considerable experience in hydro-electric work. He has made a thorough study of the problems of surge chambers, and was connected with the design of the surge chamber in the power-plant of the Los Angeles Aqueduct. As the surge chamber suggested by Mr. Freeman was an important point in the review, it was felt that Professor Durand's services were especially valuable in this as well as in the other phases of the subject. Mr. Baum has been connected with hydro-electric work in California for nearly twenty years. He was employed on the design of the first long-distance transmission line in the State, with the Standard Electric Company. He was for years the General Superintendent of the Pacific Gas and Electric Corporation, having in charge the building and operation of the numerous plants of that system, which is the largest in California. For a number of years he has acted as the Consulting Engineer of that Company on some of the largest hydro-electric installations in the West. The writer has been connected with the design and construction of hydro-electric works for fifteen years, has acted as Consulting Civil Engineer for the Pacific Gas and Electric Corporation for several years, was a member of the Board of Advisory Engineers on the Sierra and San Francisco Power Company installation, has had charge of the construction of several smaller plants, and recently acted as Consulting Engineer on the installation of the fifth unit, of 10 000 kw. capacity, of the Great Western Power Company. None of the members of the Board is a resident of the City of San Francisco, and all are without connection politically with any one in the city. There would seem to be nothing improper in the fact that the three members of this Board were named by the City Engineer in the ordinance authorizing their employment. Most engineers will agree that it was better for the City Engineer to make the selection than to leave it to the non-technical Supervisors or the Board of Public Works. As a matter of fact, no member of the Board ever conferred with any member of the City Government except the City Engineer. It would also seem that the previous experience of those

Mr. Galloway. named sufficiently justified the judgment of the City Engineer in his selections, and that the appointments were as far removed from the domain of politics as it is possible to have them.

In Mr. Grunsky's statements, the imputation is also conveyed that the various officials of the City refuse to provide for a Board of Water Supply because they all desire to "have a hand in the game". The writer is not acquainted with the opinions of any members of the City Administration, except those of the City Engineer, Mr. O'Shaughnessy, and he has stated publicly, before the Commonwealth Club of San Francisco, and also has expressed the opinion to the writer a number of times, that the proper way to take care of the construction of the Hetch Hetchy project would be to place it entirely in the hands of a Board of Water Supply disconnected from the other departments of the City's organization. Apparently, the whole procedure is at present awaiting the election for the purchase of the local water company's plant. It is quite possible that when this subject is decided, steps may be taken looking to the formation of a Board of Water Supply, but of this the writer has no knowledge. It would seem that some recognition of the public utterances of the City Engineer on this subject should be taken as an evidence of his opinion in the matter.

The writer has been a firm believer in the project of bringing water from the Sierra Nevada Mountains to the Bay districts of San Francisco, which was first definitely set forth by Mr. Grunsky. That the project has suffered various changes since the original plans were proposed, due largely to enlargement of the idea, is only natural. It is believed by the writer that the report by Mr. Freeman was an able and comprehensive document which considered thoroughly all the needs of the situation as it then appeared. Full credit must also be given to Mr. Grunsky and Mr. Manson for their earlier labors along the same line. Mr. Freeman's report was undoubtedly of material effect in securing for the city the rights for which it was struggling. The application for permits under earlier administrations had failed, and it was necessary for the officials in charge to make such compromises as they deemed expedient at the time when the Bill was finally passed by Congress. The conditions became a matter of judgment, and those who were in charge must be given credit for honesty of intentions and a knowledge of the situation as it then existed, even though the conclusion reached may differ from the ideas held by others.

Mr. Baum. F. G. BAUM,* M. AM. SOC. C. E. (by letter).—The writer notes the following statement by C. E. Grunsky, M. Am. Soc. C. E.:

"At the time the city thus voted bonds, there should have been created a Board of Water Supply, free from politics. This was not

* San Francisco, Cal.

done. The administration at that time and the administrations that have followed, would not let go. The Mayor, who names the Board of Public Works, and who practically selects the City Engineer for the Board of Public Works, the Supervisors, with the Public Service and Finance Committees, the City Attorney, the Board of Public Works, and the City Engineer, all have a hand in the game. Mr. Baum.

"There is no programme as yet for any change. Within a year there was named a Board of Consulting Engineers to pass upon the hydro-electric features of the first power installation under the unauthorized project, understood to be for 50 000 h.p. These engineers were designated by name in the City Engineer's request for their appointment. From this circumstance alone, it is to be inferred that the political management of the city's water project will be continued. Deplorable as this may seem, it is yet possible that competent management will not be too greatly hampered by political obstruction and political direction; and the hope may be expressed that, despite the unfortunate circumstances of the past, San Francisco may yet achieve an adequate and in every way satisfactory water supply."

The fact that the City Engineer did select the men, and the further fact that none of them is a politician (and does not even reside in San Francisco) should both be evidence that politics had nothing to do with the appointments. Mr. Grunsky's remark is like stating that two forces acting in the same direction could produce motion in a body in an opposite direction, both his premises being opposed to his conclusion.

The fact that there was immediate necessity for directing certain matters specifically, which had been only dealt with in general by Mr. John R. Freeman, in order to file the proper papers with the U. S. Government, was the reason for the appointments.

Mr. Grunsky's premises, his conclusions, and his inferences are therefore wrong.

E. G. Hopson,* M. Am. Soc. C. E. (by letter).—The discussion of this paper by the City Engineer of San Francisco reveals a point of view of decided interest and importance, being, in fact, the official viewpoint of the situation as it is to-day. Mr. O'Shaughnessy, who is comparatively a newcomer in official circles, undertakes to defend in a somewhat chivalrous manner many of the lapses of his predecessors by endeavoring to reconcile these with the present status and to persuade us that the action now proposed in certain official circles is the best. Mr. Hopson.

Mr. O'Shaughnessy complains that Mr. Cory's paper "consists chiefly of a résumé of matters which were thoroughly and scientifically discussed and settled"; also that "Mr. Cory attempts to revive a number of dead issues, and questions the engineering and financial policies of San Francisco as if these matters had not already been thoroughly

* Portland, Ore.

Mr. Hopson.

considered and disposed of by a number of distinguished engineers", etc. This is the crux of the entire matter, and the writer can well understand that it is at least embarrassing to the official mind to find that, even after all the elaborate investigations that have been made during the past few years and the momentous decisions that have been reached by past officialdom, the main question at issue still refuses to be cavalierly disposed of, but confronts us as if nothing had been done nor any investigations or reports had ever been made.

Thus, after all that has happened, the real issue forces its way to the front in all its baldness and crudity: Does San Francisco want any distant supply? Is the Tuolumne supply what it wants? To these questions may now be well added the minor one as to whether it is not cheaper and better to lose even whatever moneys have been spent on the Tuolumne system to date, than spend forty or sixty good dollars to save one bad one. The last issue has been created by the well-meaning enthusiasts who have led the City to commit itself to expenditures on the Tuolumne supply to an extent amounting, the writer is informed, to more than $1 000 000.

Among the smoke and dust of political and engineering controversy on this classical topic, the following points can be discerned as solid ground, affording certain though limited support. These points are:

(*a*) The practical certainty that the Spring Valley system can be acquired, if the City so desires, at a price that has now been mutually agreed to both by the City and the Company. This is a highly important feature.

(*b*) That the Spring Valley system is susceptible of developing a capacity of between 100 000 000 and 200 000 000 gal. per day, according to the authority that may be preferred. It is important to have in mind that these capacities are, respectively, about double and quadruple the present uses of water in the City.

(*c*) That if the City should acquire the Spring Valley system it would have a municipal supply sufficient to last it for the next 20, or possibly the next 50, years.

(*d*) That the quality of the Spring Valley water is admitted to be excellent.

It may be logically deduced from the foregoing axioms, therefore:

1.—That the objection of the people of San Francisco to a privately owned water supply can be immediately removed by the acquisition of the Spring Valley system;

2.—That there is ample time to consider and settle all the vexed engineering questions as to the available supply in the present system and the City's needs, long before any additional supply to the present sources will be required.

Mr. Hopson.

In the light of the foregoing, does it seem reasonable for the City to proceed at this time to commit itself to any plan leading to expenditures ranging from $37 000 000, as estimated by Mr. Freeman, to $64 000 000, as estimated by Mr. Cory and others, on an independent system that is certainly not needed to-day nor to-morrow, nor for a great many years, and which, moreover, appears to a number of apparently competent engineers and business men to be not well advised?

Mr. Cory and other thinkers of San Francisco urge that the City should pause to consider and weigh carefully the issue and not rush precipitately into these vast expenditures or do anything further to embarrass the community, already heavily encumbered, with another load of debt. They believe there is no reason for action on the Tuolumne or any other additional supply for a long time, and that the City cannot afford to commit itself either to a $37 000 000 or to a $64 000 000 proposition, much less mistake. It is urged, and the writer believes properly, that unnecessary expenditures, which are tantamount to waste, in this direction, must be reflected by curtailments in city improvements along other much more important lines.

Take, for instance, the proposition as it is advanced to-day by the advocates of the Tuolumne scheme—the City is to proceed of its own responsibility to build a metropolitan water supply system intended to serve not only San Francisco but all the bay cities. Did the City of Boston proceed to build the Metropolitan Water System of Massachusetts of its own initiative before the Massachusetts Metropolitan Water District was created, on the expectation that all the other metropolitan cities would enter at some future date and share the cost? Not that the writer is aware. Did the City of New York adopt a Metropolitan Supply sufficient for Brooklyn, Staten Island, and other communities around the mouth of the Hudson before the City was consolidated into Greater New York and all members of the metropolitan district were established as component parts of the system? It did not.

Is there any known case, worthy of being cited as an example, where a metropolitan water system has been built ahead, or independently, of the legal subordination of the several parts of the system to the whole? What would any one think of the business acumen of an individual who, living in a village community, undertook, not only to lay a pipe to supply his own house and barn with water from the near-by spring, but mortgaged his farm to pay for a system of pipes to supply all his neighbors, without taking the trouble to consult them seriously or to make any agreement with them in advance? It does not require much imagination to picture the troubles and embarrassments he would encounter in trying to pay interest on his borrowed money, year after year, or his entreaties to his neighbors

Mr. Hopson.

that they hurry up and take his water and relieve him of the crushing burden he had assumed so rashly on their account. The proponents of precipitate action on the Tuolumne supply matter are endeavoring to place the City of San Francisco in this invidious position.

The question at issue is not wholly one of cost, or the difference between a $37 000 000 Freeman scheme or one of $64 000 000, as estimated by others; nor does it consist of the difference between the estimated capacity of the Spring Valley system, as to whether it can be made to yield 100 000 000 gal., as urged by Mr. O'Shaughnessy, or 200 000 000 gal., as claimed by General Chittenden and others. The real question at issue is whether the Tuolumne proposition is intrinsically a good one, and, even if so, whether it is a good business proposition from the standpoint of the City.

This question, in the writer's judgment, is entirely unsettled, with the preponderance of evidence thus far presented against the Tuolumne scheme. The essential point, however, is that no action should be taken to commit the City to this scheme or any other scheme for at least 5 years, probably not for 10 years, after the Spring Valley system shall have been acquired, and in no event until a Metropolitan District shall have been legally organized.

Mr. Peterson.

O. W. PETERSON,* ASSOC. M. AM. SOC. C. E. (by letter).—The irrigable area of the Sacramento Valley contains from 2 000 000 to 2 500 000 acres, and that of the San Joaquin Valley 4 800 000 acres.† The aggregate mean annual stream flow into the Sacramento Basin is, in round numbers, about 30 000 000 acre-ft., and the inflow into the San Joaquin is about 10 000 000 acre-ft. The northern basin has a run-off equivalent to a depth of 12 ft. over its irrigable area, that of the southern basin being equal to a depth of about 2.1 ft.

It is impossible, however, to conserve the entire run-off. The total capacity of available reservoir sites in the Sacramento Basin is sufficient to furnish a net annual yield of more than 3 000 000 acre-ft. This supply, in addition to the summer stream flow, will irrigate about 1 500 000 acres. The remaining lands can readily obtain a supply from return waters and from wells.

No definite information is available regarding the net yield to be obtained by practicable storage in the San Joaquin Basin. It would be most unusual if a net annual yield of more than one-half of the entire run-off could be conserved in reservoirs and underground storage combined. This would be nearly five times as great a ratio of storage capacity to run-off as that given for the Sacramento Basin. This 5 000 000 acre-ft., in addition to the summer stream flow, would supply about 1 800 000 acres, or somewhat more than one-third of the entire irrigable area in the basin.

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* La Grange, Cal.

† *Bulletin No. 239*, Office of Experiment Stations, U. S. Dept. of Agriculture.

Mr. Peterson.

The general conditions of water supply in the two valleys are fairly well known. At the recent Inland Waterways Convention, held in San Francisco, Senator James D. Phelan, California's Junior Senator, is reported to have stated the situation most aptly:

"We have before us the great Sacramento Valley—too much water, and the great San Joaquin Valley—too little water. This problem ought to be solved by science."

Now, regardless of this condition of affairs in the two valleys and of its realization locally, San Francisco is proceeding to develop for use outside the basin of the San Joaquin what is probably the best reservoir site in such basin, and to withdraw thereby a supply of water sufficient for the irrigation of nearly 200 000 acres of land. This is more than 10% of the entire area in the San Joaquin Valley which can be fully irrigated. Moreover, a very large proportion of such area is the higher-lying, rolling, foot-hill land adapted to citrus fruits, so that the present value of such a tract of 200 000 acres, with full but undeveloped water supply, is really not less than $30 000 000. The author's figure, $18 924 000, is entirely too low. In time, of course, the value would be much greater.

With this (at least) $30 000 000 added to the estimated cost to San Francisco of the proposed Tuolumne River development, it appears that a fairer basis of comparison with the sources in the Sacramento River Basin would be obtained.

Mr. Rhodin.

CARL J. RHODIN,* M. AM. SOC. C. E. (by letter).—Broadly speaking, Mr. Cory's paper shows that increased water supply for the San Francisco Metropolitan District can be considered from the following sources:

1.—Further development of Spring Valley system;
2.—A supply pumped from the San Joaquin or Sacramento Rivers;
3.—Hetch Hetchy project;
4.—Other Sierra or Mountain sources.

Each of these groups has its peculiar characteristics; they are here briefly summarized.

1.—Spring Valley System.—The storage system and sources of supply of the Spring Valley Company, without doubt, should be incorporated and form an important part of any increased water supply system for San Francisco. As it stands at present, the Spring Valley system can be readily extended to provide much additional water at relatively small cost, and not only can immediate needs be met, but demands for many years can be supplied by the purchase of this property.

* San Francisco, Cal.

Mr. Rhodin.

Due to peculiar conditions in California, however, it is desirable to secure an additional source at this time for use when the economical development of the Spring Valley resources has been reached. This is due to the fact that large quantities of water in the streams of California are continually being appropriated for permanent use in irrigation. When once applied in this way, the water cannot be taken for other purposes without great economic loss to agriculture and the payment of immensely high condemnation prices by the parties desiring the water.

Furthermore, water for domestic purposes should not be taken from a territory where it can economically and readily be put to irrigation use, because the development of California depends so largely on the increased irrigation of its arid lands. Interference with this development by San Francisco indirectly hampers its own growth.

2.—Supply from the San Joaquin or Sacramento Rivers.—Popular opinion in San Francisco, generally, is not favorable to a supply from the lower reaches of the rivers. This feeling may not be warranted by the facts, but it exists, and must be considered. Popular support would be more readily found for a supply of mountain water than for a pump supply however pure it may be. Nevertheless, such a pumped supply, adequate in quantity and quality, does exist.

3.—Hetch Hetchy Project.—Under the Raker Bill, Congress has granted San Francisco the right to develop the Hetch Hetchy project, subject to certain restrictions, the most serious one being that the water requirements of 300 000 acres of irrigation land must first be taken care of. Opinion is somewhat divided as to how much water is required for the irrigation districts. Estimates vary from 600 000 to 1 050 000 acre-ft. per year delivered to the land. The proposed San Francisco supply contemplates 400 000 000 gal. daily, corresponding to about 450 000 acre-ft. per year, making a total demand of from 1 000 000 to 1 500 000 acre-ft. annually. This does not include losses through evaporation, percolation, seepage, and leakage.

The water* discharged by the Tuolumne River, at La Grange, is shown in Table 8, from which it is apparent that, to supply the total requirements, a very complete development of the water-shed will be necessary.

TABLE 8.

Water season.	Total run-off, in acre-feet.	Water season.	Total run-off, in acre-feet.	Water season.	Total run-off, in acre-feet.
1895-96	1 590 000	1900-01	2 720 000	1905-06	3 530 000
1896-97	2 440 000	1901-02	1 610 000	1906-07	3 760 000
1897-98	960 000	1902-03	1 970 000	1907-08	1 070 000
1898-99	1 330 000	1903-04	2 660 000	1908-09	2 650 000
1899-1900	1 630 000	1904-05	1 720 000	1909-10	2 080 000
				1910-11	3 410 000

* Water Supply Paper No. 299, U. S. Geological Survey.

Mr. Rhodin.

Under the Act, the burden of providing funds for construction rests on San Francisco, and it is apparent that although that city must carry the project to completion and be responsible for its success and all the contingencies, it has first claim to about the last third of the output only, the first two-thirds going to the irrigation districts.

Under these conditions, even if development costs were low, the nature of the restrictions are undoubtedly serious. As the development costs are far from being relatively low, the advisability of proceeding slowly with the project becomes very important. If it is found that Hetch Hetchy is the best available source for an ultimate large supply, San Francisco must take the risk, but it certainly is advisable to investigate most carefully whether the risks, responsibilities, and expenses attached to Hetch Hetchy cannot be avoided.

4.—Other Sierra or Mountain Sources.—From the view-point that a desirable project should not deprive irrigable areas of future development possibilities, the water-sheds of the San Joaquin River and its tributaries do not seem to warrant further investigations. It is probable that the irrigable area in the San Joaquin Valley can ultimately consume all the economically developed and regulated flow available from these water-sheds.

The Sacramento Valley probably offers better possibilities, the McCloud River, Indian Valley, Yuba River, and other projects have been investigated more or less completely, and, in the main, offer possible solutions; but, some objection attaches to them, as there may be a partial demand on the water for irrigation use. Closer study may well prove some one of these projects acceptable, and free from the objections pertaining to Hetch Hetchy.

Another project, of which comparatively little is known, appears to possess features of considerable merit; this is the Eel River project, briefly mentioned by Mr. Cory. A portion of the drainage area of the Eel River was mentioned in the investigations preceding the passing of the Raker Bill, but did not receive extended consideration on account of insufficiency in size, and lack of cost data. A larger portion of the drainage area of the Eel River could be made tributary, and a much larger supply could be developed.

The Eel River rises in the Coast Range Mountains and flows northwestward through Mendocino and Humboldt Counties, discharging into Humboldt Bay. It passes through a forest country, where the future irrigation demands are not likely to become of importance. Construction offers no abnormally serious problems. From the diversion point, the conduit, in part paralleling the Northwestern Pacific Railway, would permit gravity flow for the greater portion of the distance to Point Pedro on the north shore of San Francisco Bay. The Bay could be crossed at this point by submerged lines to its eastern or Alameda County side. The conduit might then follow the shore

Mr. Rhodin. line to Dumbarton, where the present Spring Valley lines into San Francisco Bay now cross; or a crossing could be made at a more northerly point. The total length of conduit would not differ materially from other proposed projects, and the profile is favorable, with no divides to cross.

Closer investigations may reveal obstacles in the Eel River project, but the general features certainly seem to indicate sufficient merit to warrant a more serious attention than it has received.

Although the problem of an increased water supply to the San Francisco Bay cities has been "on the carpet" for many years, the discussions and investigations which have taken place have been so clouded by political and other extraneous arguments that it is quite possible that a number of meritorious projects have been entirely overlooked.

Mr. Miller. CLEMENT H. MILLER,* ESQ. (by letter).—It is not altogether remarkable, judging from previous policies and reports, that, thus far, the discussion of this important subject has not brought forth the vital facts incorporated in the last official Government report† on the subject, made in response to the Senate resolution of October 31st, 1913, requesting certain information relative to the Tuolumne, Stanislaus, Mokelumne, and Cosumnes Rivers. The following quotations are from that report:

"Capacity of storage available for city water supply.—As shown above in this report, the storage capacity available for use of incorporated cities and towns, after allowance has been made for irrigation, is, for Tuolumne River, 550 000 acre-ft., Stanislaus River, 250 000 acre-ft., Mokelumne River, nothing, Cosumnes River, nothing." (Page 29.)

It should be noted that this allowance is based on the total available storage of 1 050 000 acre-ft., requiring the construction of twenty-three reservoirs, whereas the Freeman report contemplated the construction of three reservoirs with a storage capacity of 685 000 acre-ft., and, on this, estimated a daily available supply of 400 000 000 gal. for San Francisco.

"City water supply made available by storage.—The proper limit of municipal supply which may be made available by means of a fixed storage capacity will be determined by the most severe period for which storage is required. For the rivers under consideration and within the periods of record of discharge of those rivers, the most severe periods have been shown to be in 1897-98 and 1911-13." (Page 29.)

"The following calculation, based on the recent period of deficient run-off of Tuolumne River, indicates that for this period the Tuolumne River is not dependable, with 450 000 acre-ft. of storage for irrigation

* San Francisco, Cal.

† Senate Doc. No. 246, 63d Cong., 1st Sess., November 29th, 1913.

and 550 000 acre-ft. of storage for city water supply, for more than 230 000 000 gal. per day. Calculation follows on pages 30-31." (Page 30.) Mr. Miller.

This conclusion is based on exact data of measured stream run-off as stated during the period of known records, but records of rainfall at Sacramento are available, which show three periods of drought since 1849 more severe than the period of 1911-13, and particularly of that from 1853 to 1861.

The facts relative to quantity of water, available storage, and area of irrigable land contained in this report are not based on any assumption, and cannot be modified to any great extent, but there is good reason to question the accuracy of the assumed "duty of water".

Under the heading "Duty of Water" (page 19), is the statement: "No scientific determination of the probable duty of water in this locality has, so far as known to the Geological Survey, been made." Accordingly, for the purpose of this discussion, the irrigation duty has been assumed at 2.5 acre-ft. of water for the gross acreage of irrigable land, measured at the point of diversion from the stream.

The writer had occasion to question Mr. W. C. Grover, the compiler of the data in said report, as to his knowledge of the subject, and especially called his attention to the report* by Edwin Duryea, Jr., M. Am. Soc. C. E., Chief Engineer of the South San Joaquin Irrigation District. This report shows that, with an irrigating head of 20 sec-ft., the water used per acre per irrigation ranged from 5.8 to 15 in. (without waste), depending on the quality of the soil, a smaller quantity not being sufficient to flood the checks. The duty of water for seasonal irrigation is given as 1 sec-ft. for 80 acres, or 4 acre-ft. per acre, applied on the land from lateral ditches.

Mr. Duryea's report is also based on actual known conditions covering both the Modesto and Turlock Irrigation Districts, and indicates a required duty of water 60% greater than that allowed in the report of the Geological Survey.

As an additional allowance of ½ acre-ft. for irrigation, during the critical period of 1911-13, would have consumed the entire 230 000 000 gal. per day credited as available for the Bay Cities, it is positively certain that a storage capacity of 1 000 000 acre-ft. (if that quantity is ever provided) during recurring periods of drought will only furnish three-fourths of the quantity which will be required ultimately to irrigate the lands tributary to the Tuolumne River.

Mr. Freeman lays great stress on the fact that a municipal water supply must be proven ample beyond any question of a doubt. In view of these facts, clearly proven by the reliable data contained in the reports quoted, the Hetch Hetchy project can in no sense be con-

* *Engineering News*, September 11th, 1913.

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sidered a dependable source of supply for San Francisco and the East Bay Cities.

Fortunately, the Report of the Advisory Board of Army Engineers shows conclusively that at least two other sources are available to supply the future needs of the people, even to twice the quantity estimated as that probably required by the need of the present century.

A thorough unbiased investigation of the relative merits of these two sources would probably avoid a repetition of the disastrous blunders and lack of achievement which have been the net results of the efforts of the past twenty years to solve the water question.

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RUDOLPH W. VAN NORDEN,* M. AM. SOC. C. E. (by letter).—This paper has been commented on, and criticized by, engineers, either with extreme favor, or with disfavor amounting to condemnation. Favorable criticism has emanated from those engineers who are familiar with the San Francisco water-supply problem, but who have not contributed to its solution; also from those who, at some time, have contributed to the study of the problem, but are not now engaged in the development of the Hetch Hetchy scheme to the exclusion of all others. The latter group naturally covers the majority of engineers more or less familiar with this subject. The adverse criticizers comprise only those engineers who are engaged on the development of the Hetch Hetchy plan, and are thus not interested in any consideration or argument which might jeopardize the fulfilment of that project.

Mr. Cory's paper cannot be considered as a cut-and-dried academic technical article of the orthodox variety, which engineers have been taught to expect in the publication of technical societies. From this view-point there is no doubt that the adverse criticism by Mr. O'Shaughnessy is in order; but the time has long since past when papers and their discussions, having as a purpose the accomplishment through engineering science of some great economic improvement, can be limited to mathematical formulas, available data, and deductions from reasonings within narrow limits, where such reasonings are based on premises which are but sage theories, in reality little better than guesses.

Engineering science is becoming recognized more and more as of infinite breadth, rather than infinitely circumscribed by rules and theories. Economic questions, in themselves apparently outside of the radius of action of the civil engineer, questions of law and personal rights, equally foreign to the old-time ideas of the duties of the civil engineer, all enter into the solution of a problem so complicated and so vital to a commonwealth, as is the water supply in question—not as foreign intrusions, but as essential parts of an engineering problem. Civil engineering, in its highest sense, does not, alone, mean the

* San Francisco, Cal.

mere design and construction of dams, tunnels, bridges, or power-plants—given unlimited capital and unlimited time, almost anybody can accomplish such things—but it does mean the selection of ways and means of doing the greatest good for the advancement of mankind with the greatest lasting economies. Mr. Van Norden.

In the San Francisco water-supply work, much deep and thoughtful research has been made. It is not true, as has been often stated, that the people's money spent in this work has been wasted. There has undoubtedly been much time and some money spent in "offside play", and irrelevant actions with political savor, but this has had its value in awakening public interest to action.

The problem, as attacked by Mr. Grunsky, the City Engineer in 1902, was handled, within the limits of cost and time available, in a masterly manner. For the object in view, as it was then understood, Mr. Grunsky's report was a sound beginning; but time has proven that there were lacking, and unknown at that period, the very elements which are shown to exist by the study in Mr. Cory's paper. As economic conditions have become better understood and realized, as the community has developed, and as settlement in the State has increased, much work toward arriving at a basis for obtaining proper results has been done. This has all been creditable, but the problem is a far bigger one than was at first realized, and all this preliminary work has been necessary to obtain insight into the matter.

Finally, the plans which have now been advanced for a proper development of a water-supply system were worked out with the object of coping with the problem in its broadest form. Theoretically, this has been a step in the right direction, but the criticism which has been freely offered is that the gauge adopted has not been broad enough. In order to make the suit fit, the cloth should be selected of ample breadth. In this case, the attempt seems to be to use the cloth at hand and accept a suit which it will make, with the grave possibility that it will not fit, or will be too tight.

The day is past when one engineer can determine the solution of a broad engineering problem by stating that, in the wisdom of his opinion, thus and so are truths, and expect the matter to be settled without further proof. The statement has been made in the criticism of Mr. Cory's paper that the plans for the development of a San Francisco water supply, engineering, economic, industrial, and financial, have been carefully made and settled, and, as an excuse for this statement, a further suggestion is made that this has been decreed by engineers of prominence.

There is ample evidence that a very considerable quantity of creditable and serious preliminary work, as a means toward an end, has been done, but Mr. Cory's paper goes to show that the pure and

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proper engineering work necessary for the solution of this problem is by no means completed. The paper illuminates the many pitfalls to be encountered, and points to the invisible angles from which views of the problem must be made, without the consideration and treatment of which all previous efforts toward its eventual consummation must be nullified. If this is true, and many engineers believe it to be so, then Mr. Cory's paper is a pure engineering document of the highest order.

Within the last three years, the present civic administration, through the city engineer and the city attorney, has promoted a number of public enlightenment campaigns, or, what might better be termed, a single campaign with intermittent periods of great activity. The noticeable feature of this campaign throughout has been, not to instruct the populace as to its efforts and accomplishments in the preparation for the development of the best plan for a water supply as a broad engineering and economic proposition, but, rather, as to its plans for a reasonable engineering development of the Hetch Hetchy project. The administration has felt itself justified in taking this attitude from the fact that some years ago the City of San Francisco purchased reservoir sites on Cherry Creek and at Lake Eleanor, spending in this purchase considerably more than $1 000 000, and, in 1910 the people voted to bond the city to the amount of $43 428 000, for the construction of the Hetch Hetchy project. Thus this project has been tenaciously upheld as the "best development possible", with little regard to the feasibility of other suggested plans, some one of which might fulfil the conditions to be met in a more satisfactory manner at a less cost to the people.

Much that has been said in this campaign of education has been sound, but there has also been much said with the intent of befogging the issue, and many statements have been made which could not possibly have any basis in fact. The right statements, and those which it would be difficult to substantiate, become apparent on examination to any engineer or investigator, but the many vital points which have been left out and might seriously cloud the appearance of superiority of the Hetch Hetchy source over some other have generally formed grounds for contention, and have resulted in the deadlock that seems to exist at the present time.

What should have been discussions of interest and value have generally been vicious attacks of a personal nature from both sides. Nor in the discussion of this paper have personal aspersions been absent; and these are the things which are not only out of place in a scientific publication, but are inexcusable.

There have been a good many engineers who have become familiar with the situation in a general way, and who would gladly join in discus-

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sions with the object of arriving at an absolutely true and unbiased basis for the correct solution of the problem. Of these, some are entirely independent and free from any interests touching this water-supply situation, others are more or less interested through clients, and still others are more directly interested; but, unfortunately, all the engineers thus situated, who have not made some expression of their studies or opinions, are divided into two classes. In the first class is the man who does not wish to take issue with the administration, because he feels that he may jeopardize his chance of working into the development in a professional capacity. In the other class is the engineer who does not wish to be accused of being in the employ of some interest whose plans conflict with those as announced by the city engineer.

Throughout this campaign all those who have dared to offer any opinions contrary to those of the administration, or to disagree with, or to take any action which would block the plans of, the administration in its procedure with the Hetch Hetchy project, no matter who they are, or what their standing, have been flatly accused of being in the wrong from some ulterior motive. This accusation has generally taken the form of statements to the effect that such persons were working in the interests of companies, or individuals, with plans for water-supply developments inimical to that of the Hetch Hetchy, and that such persons were receiving pay for their contrary opinions. There was a long period when any engineer daring to express a contrary opinion was said to be in the employ of the Spring Valley Water Company.

Such campaigning has undoubtedly had an immediate desired effect, but, in the long run, it has caused many people to think, and in the end the administration cannot but feel the ill effect of these short-sighted statements, where there has been so little foundation in fact.

In proceeding with the plans for the Hetch Hetchy project, the administration has taken its position—which it has apparently determined not to compromise—on the basis of two engineering reports. These are: the report of John R. Freeman, M. Am. Soc. C. E., which includes several reports on alternative projects by other engineers, and the report made on the order of the Secretary of the Interior by an Advisory Board of Army Engineers. Since the acceptance of these reports, the city engineer has done much work in amplifying and recasting the information embodied, with respect to the Hetch Hetchy project, until his part of the work is probably more valuable than the contents of these two reports combined.

In the earlier work done by the office of the city engineer, namely, during 1902 and 1908, a project was outlined for the immediate development of a supply of 60 000 000 gal. daily, with provision for

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doubling that quantity. On this basis a system was designed which would compare favorably in cost with that of all other projects, and held no very serious engineering difficulties.

These plans represented sound practice in engineering construction. Furthermore, the water-shed and storage system proposed offered an ample reserve which took cognizance of all complications, and would satisfy adverse claims to water.

The present administration realized the necessity for a water system capable of delivering 400 000 000 gal. per day. This requirement presents a very different engineering and economic problem, compared with which, the first plans are as mere child's play.

With the determination of the necessity of a greater quantity, there was immediately presented an array of possibilities for the development of some other water supply, which, under the smaller draft of the earlier plans would be uneconomical, and hence unfeasible, but, under the new plan of increased output, might prove to be much superior to the Hetch Hetchy project.

These possibilities were apparent to every engineer and investigator who knew anything about the water situation, but, as has been said before, the inflexible stand taken by the administration precluded discussion, or even opinion to any extent.

The Freeman report is, on its face, a justification of the use of the Hetch Hetchy Valley and the Tuolumne water-shed. It is the suit made to fit the cloth. As a mass of statistical information, it is a splendid compilation, and, considering the comparatively meager data on which it was built, is an apparently strong presentation of the subject. The writer has discussed with many other engineers the general engineering plan adopted in the Freeman report, for the purpose of determining the soundness of the reasonings held therein and the plausibility of cost estimates, and to obtain, if possible, suggestions for improving the plan, or lowering the cost. Though there are diverging opinions on these points, the writer believes that the general plan for pressure tunnels, as outlined in the Freeman report, is visionary, and that the estimates of cost are entirely too low. In this opinion he has been upheld by other engineers. The writer does not take the stand that the general plan is untenable, as it is possible to give much latitude in the final designs and determinations, but unless this is possible and unless the foregoing statements are disproved, the entire plan for the development of so great a transportation as 400 000 000 gal. of water per day becomes practically unfeasible in the face of possible supplies from other sources.

The writer is strongly opposed to the construction of unbalanced pressure tunnels. By this expression is meant, a tunnel operating under hydrostatic pressure where the mass of ground over and around

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the tunnel is depended on to resist the tendency toward bursting, and where there is no hydrostatic pressure from the outside which would tend to balance the pressure from the inside. A balanced and hence proper use of pressure tunnels is illustrated on the Catskill Aqueduct in the crossing under the Hudson River and the tunnel under the City of New York.

In the unbalanced pressure tunnel there is always a tendency toward leakage, depending on the homogeneity of the lining and surrounding structure. This leakage will work toward the surface, and the pressure follows in time. Theoretically, there will come a time when some layer of pressure close to the surface will overcome the resistance of the ground material, and a blow-out or slip will occur, only to be followed by another slip farther back. This action will continue toward the source of pressure until the entire structure of the ground fissures, or gives way bodily.

There have been, to the writer's knowledge, four pressure tunnels in California, all of which have failed in a greater or less degree, and in one manner or another. The two most notable and costly examples of such failures were the inclined unbalanced pressure tunnels in the Sand Canyon Siphon on the line of the Los Angeles Aqueduct. These tunnels were 9 ft. in diameter and were lined with plain concrete having an average thickness of 12 in. They were cut through granite. Both were inclined from the grade of the gravity conduit, these inclines being in the same direction as the respective slopes of the canyon sides, but with steeper inclinations. They were connected at their lower ends by a riveted steel pipe, 9 ft. in diameter, this pipe being run into and concreted in the horizontal portals of the pressure tunnels. The maximum thickness of ground, measured normal to the axis of the tunnels, was about 500 ft. in one case, and more than 100 ft. in the other. Both of these tunnels blew out shortly after they were filled with water. The symptoms were, first, a slight leakage, then a small movement, followed, in one case, by the tearing out of the whole mountain side. It was necessary to replace these tunnels by an all-steel siphon. Aside from the original cost of the work and the cost of rebuilding, the absolute stoppage of flow and use of the aqueduct, had it been in regular service at that time, would have been fully 2 months. The writer cannot conceive of any other result which might have been expected from the use of such conduit construction.

In the plan for the Hetch Hetchy development, there are long stretches of pressure tunnel. In much of this distance, blow-outs or breaks would probably be impossible or unlikely, but there are undoubtedly many places where disasters of this sort might be looked

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for. One such break as was experienced at Sand Canyon would interrupt the flow of water for weeks, or even months.

Between the west side of the San Joaquin and the east rim of the Santa Clara Valleys, there are proposed, according to the Freeman plan, two pressure tunnels with an aggregate length of 30.7 miles. The first of these is to be 27.5 miles long. Geological studies of the country through which these tunnels are to pass show well-defined faults. A break or slip in the ground surrounding one of these pressure tunnels might be the cause of endless disaster to the system; and the emptying and refilling of the conduit, even for inspection or repair, might cause a most serious water famine in San Francisco by the interruption of the supply. Particularly would this be so if the present and proposed reservoirs of the Spring Valley Water Company, which are included as an integral part of the Freeman plan to act as stand-by storage, are not available for use in this connection; and, at the present writing, such an eventuality would seem to be probable, as the purchase of the Spring Valley system by the City of San Francisco has failed to carry at two elections.

The estimates of cost of the Freeman plan were based largely on unit cost data obtained from the work on the Los Angeles Aqueduct, but at a time prior to the actual completion of the work and the commencement of operation of that system. The writer believes that the use of these Los Angeles cost data in the Freeman report was premature and inapplicable to the Hetch Hetchy project; that in many points actual costs would be greater than those assumed; and that the total cost of the work, if it were possible to complete to successful operation in accordance with the Freeman plan, would sum up greatly in excess of the totals given in that report.

To sum up the preceding remarks, the Freeman report is by no means a complete solution of the problem to deliver water from the Tuolumne water-shed, either from an engineering or an economic standpoint, but it serves to disclose the magnitude of the problem of the development of the Hetch Hetchy project and to emphasize the complications unseen by the casual observer, and furthermore, to point out, not the ease with which the project may be constructed, or its cheapness, but rather, the tremendous difficulties of making a source —quite suitable for a small supply of 120 000 000 gal. daily—equal to the duty of the supply and transportation of 400 000 000 gal. daily, even for distant future consideration, which is a figure at the extreme outside limit of the water-shed and storage possibilities, economic conditions not considered, and with little or no factors of safety.

The report of the Advisory Board of Army Engineers is an elaborate and very fair document, considering the variable sources of the data, many of which were of little value, and withal, comparatively meager, on which it was compiled. On a favorable interpretation

of this report for the Hetch Hetchy project, the administration has placed much stress. Mr Van Norden.

The report and subsequent extensions were used with good effect before Congress in assisting the passage of the Raker Bill. Few people have taken the trouble to read this exhaustive report through, much less to study it—an unfortunate condition, because it is broad in its conception, and shows much care and thought in detail.

The criticism of those who are familiar with it is that the administration, in using it, is careful to quote only those paragraphs, sentences, and even clauses, which will react favorably to the Hetch Hetchy plan. To study the report gives one the impression of a broader understanding of the subject by the Board of Army Engineers than the interpretation thereof which has been consistently maintained by the administration.

In placing those data before the Army Board on which to base a report, the city engineer was required to give, not only all data available on the Hetch Hetchy project, but also on the various other projects offered. This latter requirement was a difficult one, because the administration, pledged to the development of the Hetch Hetchy project, possessed only preliminary and incomplete reports already published on some of the alternative projects, and practically no data and little information on the remaining alternatives. It thus became necessary for the city engineer, within a very limited time, to examine these projects, collect data, and make reports of their possibilities, methods of construction, and costs. Attempts on the part of the city engineer were made to proceed with this Herculean task, which would require months where days were available. The results were naturally abortive and of little value, but served to accomplish in a measure the requirement of the administration, namely, to cause the Hetch Hetchy plan to stand in a superior light in comparison with all other suggested plans and projects.

As an example of the frailty of these reports made by the city engineer: two of the alternative projects suggested would be natural flow aqueduct systems, in many ways similar to the Los Angeles Aqueduct. One of these, known as the McCloud project, proposes to take a part of the natural flow from the McCloud River without the requirement of initial storage reservoirs as a primary requisite. The proposed line is to be a gravity-flow aqueduct, partly in canal and partly in tunnels, with the exception of the crossing under Carquinez Strait, which would be a pressure tunnel similar to that of the Catskill Aqueduct under the Hudson River. Throughout its length of about 200 miles, this aqueduct would pass through a very rough and mountainous country—the eastern range of the Coast Range Mountains, bordering the west side of the Sacramento Valley. The greater part

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of this territory is uninhabited, and the aqueduct location, at the present time, is largely unapproachable by vehicles, because of the absence of roads.

In order to obtain data for the required report on this specific project, the writer was informed that the city engineer sent out a party which made a rapid topographical survey of the reservoir site at the proposed location of the diverting dam in the McCloud River. This required several days. No criticism could be offered for this part of the city engineer's work, except that the work and time spent on this part of the project were rather unnecessary. Another party, so the writer was informed, attempted to cover the 200-mile stretch of the proposed aqueduct line in an automobile, reaching its supposed position by means of an occasional intersecting road, and with the assistance of such portable instruments, including a camera, as would be used in rough reconnaissance work. Three days are said to have been spent in examining this stretch of country to obtain first-hand data for the compilation of the city engineer's report. The strenuosity of this work may be better realized when it is known that, for the greater part of the distance, there are no topographical maps, or accurate maps of any kind, in existence.

The writer is more or less familiar with the country through which this proposed aqueduct is to pass, and fails to conceive how any engineer, no matter how great his ability or experience, could properly cover this line and obtain data of any value whatsoever for a report in ten times the period taken by the party in question, even had he been over the country on some previous occasion. The proof of this contention is found in the reading of the report and the observation of photographs taken, purporting to be points on the aqueduct location, which are, in reality, several miles from its true position. As a further proof that a fair and unbiased presentation of this project was not made, the writer is not unconvinced that a better and cheaper means of bringing the McCloud water to the Bay Cities would be, not by a gravity aqueduct, but by pipe line through the open Sacramento Valley, operating under a static head only sufficient to overcome the friction head of the pipe. This feature was not treated seriously.

The writer is not in a position to state flatly that the McCloud project is superior in the broad engineering sense, that is, good engineering construction at lower comparative cost, better economic conditions, more equable personal rights, and the greatest good to the greatest number, than is the Hetch Hetchy plan properly developed after exhaustive study; but it certainly represents many advantages over points now standing in the way of the Hetch Hetchy project consummation, and is, without doubt, worth very careful and unprejudiced engineering study.

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The preceding statement is made absolutely without prejudice, as the writer has no interest for or against any plan for a water supply, outside of a desire to see the problem properly solved.

Those features which circumscribe the use of the Tuolumne River for municipal supply, and otherwise impose conditions which enter vitally into the solution of the problem as a whole, that is, the question of economics and personal rights, have been discussed in a limited way in connection with Mr. Cory's paper. The writer believes that there remain ample grounds for further discussion in a broader and fairer-minded manner on both sides, and these points must be all settled before plans for detailed construction can proceed intelligently.

Without going into any of these questions, or making the attempt to supplement Mr. Cory's researches along these lines, there is, however, one point which the writer desires to emphasize:

The City of San Francisco, through its representatives, made a hard fight before Congress for permission to construct reservoirs and conduits to carry water from the Tuolumne water-shed. This fight, which the people of San Francisco as well as the entire country were led to believe was a desperate contest between justice and right, on the part of the people, as against powerful and sinister interests whose only desire was to thwart the Commonwealth of San Francisco for exploitation and personal gain, resulted in the passage of the Raker Bill. This bill was, indeed, a hollow victory, for the adherence to its requirements loads upon the City added responsibilities of great magnitude. It also jeopardizes the water supply by limiting its output in years of low water yield far under the point of maximum use.

Coincident with the promotion of the passage of the Raker Bill, an independent report on the subject of the use of the Tuolumne water by San Francisco was asked for by the Secretary of the Interior, in response to Senate Resolution No. 191, from Dr. George Otis Smith, Chief of the United States Geological Survey. This report, in conjunction with a supplementary report by H. H. Wadsworth, M. Am. Soc. C. E., for the Advisory Board of Army Engineers, in response to the foregoing Senate resolution, is entitled, "Letter From The Secretary Of The Interior Transmitting, In Response To A Senate Resolution Of Oct. 31, 1913, Certain Information Relative To The Tuolumne, Stanislaus, Mokelumne, and Consumnes Rivers In California".

The major premise on which to base any reasoning as to the water available for a municipal supply from the Tuolumne River, based on prior irrigation rights to the water, is the minimum seasonal water allowance necessary to irrigate the lands of the Modesto-Turlock irrigation districts. Several engineers familiar with irrigation requirements in California have placed this allowance at from 4 to 2½ acre-ft. per acre per year, net, that is, measured on the land to be irrigated.

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The larger figure, 4 acre-ft., was determined by Edwin Duryea, Jr., M. Am. Soc. C. E., from elaborate tests and measurements of water used over both districts, and is an average which he considers must be adopted as a safe minimum.* A figure of 2¾ acre-ft., net, has been given by Burton Smith, M. Am. Soc. C. E., formerly Engineer for the Turlock District, and the lower figure of 2½ acre-ft., net, has been mentioned by Messrs. Grunsky, Cory, and others, and is that adopted by Dr. Smith in his Senate report.

The board of Army Engineers, as shown in Mr. Wadsworth's report, have adopted a smaller water duty for irrigation, allowing 2½ acre-ft. per acre to be irrigated, measured at the point of diversion, and hence allowing nothing for evaporation and seepage in canals and ditches. In this difference lies the variance in the conclusions of Dr. Smith and Mr. Wadsworth. The report of Dr. Smith sums up as follows:

"The following calculation, based on the recent period of deficient run-off of Tuolumne River, indicates that for this period the Tuolumne River is not dependable, with 450 000 acre-ft. of storage for irrigation and 550 000 acre-ft. of storage for city water supply, for more than 230 000 000 gal. per day."

Here follows an elaborate calculation of the actual quantities of storage, in place and as the result of draft by irrigation and by the municipality, beginning with July, 1911, when it is assumed that all storage reservoirs would have been full. An equation quantity, x, is used to denote the monthly draft for city use. Evaporation during the late summer months from the reservoirs devoted to city supply is placed at 25 000 acre-ft. This calculation concludes with the following sentences:

"The condition of storage in the City reservoirs on January 1st, 1914, would be represented by 712 592 — 90 000 (accrued evaporation) — $29x$." "If we assume that the reservoirs are empty on that date, then we have the equation, 712 592 — 90 000 — $29x = 0$." "Then $x = 21\,469$, the amount of water, in acre-feet per month, which the city might have drawn during the period from August 1st, 1911, to January 1st, 1914." "This amount is equivalent to 233,000 000 gal. per day draft over the entire period of 29 months."

There were only 200 copies of this document printed, and most of these were distributed in the Senate, a copy being placed on the desk of every Senator. The fact that very few people have ever seen this report, and that its mention has been consistently avoided by newspapers and by those who have been actively interested in the furtherance of the Hetch Hetchy project to the exclusion of the consideration of all other plans, is interesting.

* "A Study of Irrigation Heads in the Modesto and Turlock Irrigation Districts, California." *Engineering News*, September 11th, 1913, p. 502.

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Under the present condition of affairs, the writer is of the conviction that the City of San Francisco cannot succeed in developing its future supply of water from the Tuolumne water-shed, or from any other undeveloped water-shed; but with the information and experience now at hand, let the city engineer begin anew, with the broad purpose of exploring every available source and method of supply, equally and fairly, without fear or favor, forgetting for the moment the money already spent and the rights acquired for developing the Hetch Hetchy. A solution satisfactory to the majority can then undoubtedly be found.

Mr. Smith.

BURTON SMITH,* M. AM. SOC. C. E. (by letter).—On page 80 the author assumes that 1 500 000 acre-ft. of storage can be provided, in about forty reservoirs scattered over the Tuolumne water-shed, at a cost of $30 per acre-ft.; and that the entire storage system can be operated with an 80% over-all efficiency. On these assumptions the basic estimates of yield and cost of the Hetch Hetchy project are deduced. Conclusions thus reached are nothing less than dangerous.

The writer, until recently and for a number of years, has been Chief Engineer of the Turlock Irrigation District, and has thus had occasion to examine most carefully and critically all storage possibilities in the entire Tuolumne water-shed—doubtless more exhaustively than any one else has done. Stated briefly, the results of these investigations are that, at most, 1 000 000 acre-ft. of at all practicable storage possibilities exist, and that the average cost of any such quantity would exceed considerably $40 per acre-ft.

These figures generally agree with those of the United States Geological Survey, as given by the Director, Dr. George Otis Smith, in his report on this subject made in accordance with the Senate Resolution of October 31st, 1913.†

On page 83 the author makes some quantity estimates which are based on 1 500 000 acre-ft. of storage—which, as just stated, is far too great—and various irrigation water requirements. Dr. Smith, in the report just mentioned, after stating that the duty of water in that locality is not yet fixed, assumes it at 2½ acre-ft. per annum for the gross acreage measured at the point of diversion from the river. This is undoubtedly lower than (in accordance with terms of the Raker Bill) the Secretary of the Interior of several years hence will determine as the water needs of the land. Fortunately for all parties concerned, it will then be a matter of supreme indifference what the city authorities or the farmers think now or will think then on this point. The only thing of any interest will be the opinion of the Secretary of the Interior of that time, based on experience. Never-

* Oakdale, Cal.

† Senate Doc. No. 246, 63d Cong., 1st Sess., November 29th, 1913.

Mr. Smith. theless, each can now entertain himself by speculating on what that Secretary's decision will be; and the writer's judgment is that, at the very least, it will be 3 and more probably 3½ acre-ft. per acre.

The writer would also be more than deeply interested in a system of operating the storage of such a quantity and character with an efficiency of 80%, or anything near it.

In short, the author's estimates in these particulars are quite too liberal.

Mr. Sloan. C. E. SLOAN,* Esq. (by letter).—Notwithstanding the author's full though concise presentation of this subject, and the interesting discussions which the paper has brought forth, several obvious and important questions are suggested, but are not answered. Some of these have been hinted at in the discussions; others have not.

The title of the paper suggests the first, and perhaps the most important, of these questions, so far as the people of San Francisco are concerned, that is, if this so-called Hetch Hetchy water supply is a water supply for the San Francisco-Oakland Metropolitan District, why is it that San Francisco is acting alone and bearing all the burden?

If it is to be a water supply for the Metropolitan District, why has not that District been formed, and why should not the $45 000 000 in bonds, voted some years ago by San Francisco for a water supply, justly rest on all the Metropolitan District? At none of the many stormy sessions held within the past 10 years, during which Hetch Hetchy has been argued *pro* and *con*, has any real effort been made to establish a Metropolitan District. Why? Principally because the men behind the Hetch Hetchy project are pretty well satisfied that an election to form such a district would be overwhelmingly defeated in the East Bay cities, unless San Francisco would not be able to control the District, in which latter event the Hetch Hetchy project might very well be abandoned.

Then why the proposal for a supply sufficient for all the Bay cities? The Grunsky project in 1902 was for only 60 000 000 gal. daily. The answer is this:

At the time of revoking the "Garfield Permit" by the Secretary of the Interior, the City of San Francisco was instructed, among other things, to investigate all other sources of supply available, and to lay before the Department of the Interior the results of this investigation. An ample supply for San Francisco alone would be 200 000 000 gal. daily, approximately five times the consumption in 1915. From the report of the Board of Army Engineers it will be seen that there are several sources which will furnish such a quantity, and the report clearly points out the fact that the needs of San Francisco

* San Francisco, Cal.

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can be supplied for many years to come without any water from outside sources. Numerous engineers have reported on the resources of the Spring Valley Water Company, and almost all agree with the Army Board that, by developing these sources to their ultimate capacity, San Francisco will not require water from an outside source until 1965. The report of the Board of Army Engineers, excluding half of the possible development of the Peninsula coast streams, places this date at 1947. This being true, it is obvious that a supply for San Francisco proper, from outside sources, of twice 200 000 000 gal., or 400 000 000 gal. daily, is not only a very expensive luxury, but a foolish undertaking for many decades to come.

As stated in the Army Board reports, there are several excellent sources good for 200 000 000 gal. daily. There are only three good for 400 000 000 gal. daily—the Sacramento River, the McCloud River, and the Hetch Hetchy. Nothing simpler: all but these three were at one stroke absolutely eliminated by raising the requirements to 400 000 000 gal. daily. To justify this large demand, only the Metropolitan Water District was needed and suggested—but not formed.

San Francisco is particularly fortunate, compared with the remainder of the prospective Metropolitan District. A careful study of the population curves, water consumption, and immediately available supplies of the Metropolitan District, shows that San Francisco can do without a mountain supply for 50 years, but such is not the case with the East Bay region and Marin County; these must look to some outside source for their water in the very near future. It would seem, therefore, that these communities might well pool their needs and join in a solution of their problems.

The various sources of supply suggested in Mr. Cory's paper are considered with reference to a supply of 400 000 000 gal. daily. Just why he should have set the same minimum quantity as the proponents of the Hetch Hetchy project before Secretary Fisher he does not explain. His careful study of the question is convincing that this quantity of water is unnecessary for the wants of San Francisco for possibly all time to come.

Assuming, however, that 400 000 000 gal. daily is the ultimate demand of the Metropolitan District, the author finds that San Francisco would require only about 100 000 000 gal. daily in, say, 1960, and 200 000 000 gal. daily about 2000 A. D., the remainder being wanted by the rest of the Metropolitan District. If this fact were more generally known, the agitation which has been carried on for the past 10 or 12 years regarding Hetch Hetchy would practically cease, so far as San Francisco is concerned. As no effort has been made by the rest of the District to obtain an outside supply, it should be realized locally that San Francisco is ill-advised in planning to

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bring in a supply of which she can never hope to use more than one-half; for which (on a basis of assessed value) she will be compelled to pay about 70% of the total cost, and very probably all the costs; and the water of which she can only sell to other municipalities, under the terms of the Raker Bill, at actual cost.

However, it does seem that an outside source of water must be obtained for a Metropolitan District (exclusive of San Francisco), and that shortly. Without the financial backing of San Francisco, the Metropolitan District could not assume an obligation of bonds sufficient to obtain water from Hetch Hetchy. From the report of the Board of Army Engineers it will be seen that the quantity of water required for the East Bay region and Marin County is a great deal less than 400 000 000 gal. daily—indeed, about 200 000 000 gal. daily at most—and that this can be had from several different sources at considerably less cost than from Hetch Hetchy. Any of these sources could be financed by the remainder of the Metropolitan District.

Notable among these, as pointed out by Mr. Rhodin in his discussion, is the Eel River supply.* Complete cost estimates on this source are not available; two reports, however, have been made public: one by Messrs. W. R. and N. A. Eckart to the Snow Mountain Water and Power Company, and one by Mr. George S. Nickerson to the President of the Modesto Irrigation District Water Users' Association. Mr. Nickerson's report is on a combination of the Southern Eel River-Putah Creek water-sheds, thereby furnishing a supply of 400 000 000 gal. daily. This would necessitate numerous expensive tunnels and the mixing of the waters of Eel River with those of Clear Lake, which latter waters, as stated in the report of the Board of Army Engineers, is not excellent. The water-shed of the South Fork of the Eel River is in Lake and Mendocino Counties, and almost wholly within the Stony Creek National Forest Reserve.

The territory tributary to Gravelly Valley Reservoir contains 268 sq. miles, and that tributary to Cape Horn Reservoir is 58.5 sq. miles, a total of 326.5 sq. miles. The rainfall in this territory is abundant:

Mean seasonal	43.51 in.
Maximum seasonal	78.62 "
Minimum seasonal	24.99 "

The ratio of maximum to mean rainfall, therefore, is 162%, and of minimum to mean rainfall, 51½ per cent.

The seasonal mean run-off is 26.81 in., and the minimum run-off is 6.90 in. Thus, the ratio of mean run-off to mean rainfall is 55% and of minimum run-off to minimum rainfall is 27.6 per cent.

* See also Report of the Advisory Board of Army Engineers, p. 83.

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The Gravelly Valley Reservoir site is an excellent one. With a dam 150 ft. high, this reservoir would impound 70 000 000 000 gal., or 15.03 in., over the tributary water-shed. This would give an available water supply (during the dryest known period in the past 38 years) of 190 000 000 gal. daily, which could readily be increased to 200 000 000 gal. daily, by taking advantage of the available large storage reservoirs in the East Bay and Marin County regions.

The elevation of the intake of this system would be approximately 1 000 ft. above sea level, and, by following the proximity of the right of way of the Northwestern Pacific Railway Company, a gravity system could be constructed. The hydraulic gradient of this system would permit of the use of light-weight steel pipe or reinforced concrete conduit.

By crossing San Francisco Bay at or near McNear's Point, which crossing offers no great difficulties, the cost of the system would be obviously low. Furthermore, the waters of this water-shed are not now, nor in the future will they ever be, required for irrigation purposes.

The joining of the East Bay regions and Marin County in some such project, and keeping quite apart from San Francisco proper, is perhaps quite as possible as the carrying out of the Hetch Hetchy project along the present proposed lines.

Mr. Duryea.

EDWIN DURYEA, JR.,* M. AM. SOC. C. E. (by letter).—On page 151 Mr. Clement H. Miller states as follows:

"* * *, the irrigation duty has been assumed [by the Geological Survey] at 2.5 acre-ft. of water [per year per acre] for the gross acreage of irrigable land, [the water being] measured at the point of diversion from the stream."

Mr. Miller also states, with reference to the report of the writer as Chief Engineer of the South San Joaquin Irrigation District:

"This report shows that, * * * the water used per acre per irrigation ranged from 5.8 to 15 in. (without waste), depending on the quality of the soil, a smaller quantity not being sufficient to flood the checks. The duty of water for seasonal irrigation is given as 1 sec-ft. for 80 acres, or 4 acre-ft. per acre, applied on the land from lateral ditches.

"Mr. Duryea's report is also based on actual known conditions covering both the Modesto and Turlock Irrigation Districts, and indicates a required duty of water 60% greater than that allowed in the report of the Geological Survey."

Mr. Miller is in error in drawing his conclusion from the so-called Duryea report. The 5.8 to 15 in. per irrigation quoted by him are specific measured instances of actual use; but the depth necessary depends largely on the skill and care with which the land has been prepared

* San Francisco, Cal.

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for irrigation or "checked-up", and on the skill of the irrigator in applying the water; and the depth used too often has no relation to that needed, but, instead, is the maximum which can be obtained, even up to depths harmful to the land.

As stated in the abridgment,* the general conclusions of Mr. P. C. Berkefeldt's report to the writer are as follows:

"For fairly sandy soil and sandy soil, crop alfalfa * * *. For the average run of the land to be irrigated (*i. e.*, the average grade, length, and width of checks, etc.), it is best to use not under a 15-sec-ft. head for a time of about 20 minutes to a half hour per acre; depth of irrigation to be from 4 to 6 in., * * *."

As an average for all the lands in such districts as the Modesto, Turlock, and South San Joaquin, the advisable number of irrigations is believed to be six per year, which, combined with the depth of from 4 to 6 in. per irrigation, stated in the Berkefeldt report to be advisable for alfalfa, is equivalent to a depth per year on the net area irrigated of from 2 to 3 ft., measured on the land. As the result of all the writer's study in connection with the design of the South San Joaquin irrigation system, he decided that the proper average depth for the irrigation of alfalfa is 4.5 in. per month, or 2.25 ft. per year, measured on the net area irrigated.

To provide for a possible "peak use" of the water, and to meet demands for a very ample supply, the Supply Canal and the Main Distributary Canal of the South San Joaquin system were proportioned to deliver 6 in. net water per month on the net area to be irrigated, after all probable losses from absorption and seepage, and without encroachment on the safe "free-board" of the canal banks; and, to provide for "rotation", the Branch Canals were proportioned to deliver the 6 in. each month when in use only about two-thirds of the full time, and the District Ditches when in use only a few days during the month.

The 4.5 in. per month, or 2.25 ft. per year, measured on the net area, are for alfalfa alone. Hardly any other crops require as much water, and in nearly all large irrigated districts a considerable and increasing proportion of the total area is in other crops. Hence, in the writer's opinion, the advisable average use of water in such large irrigation districts should be materially less than 2.25 acre-ft. per year per acre of net area. This is as measured on the lands irrigated that year, presumably about 80% of the gross area of the irrigation district; referred to the gross area, the advisable depth would be materially less than $2.25 \times 0.80 = 1.80$ ft. per year.

Of course, to supply these net depths of water on the lands to be irrigated, enough additional water must be diverted from the river into the headworks of the system to provide for all absorption and

* *Engineering News*, September 11th, 1913, p. 503.

seepage losses from the canals between the river and the irrigated lands. In the design of the South San Joaquin irrigation system, it was concluded that, at the most, 36% of the water diverted from the river might be lost by absorption and seepage from the canals and ditches—that part lost from the Distribution System tending to supply sub-surface irrigation more or less, however, and hence being only a partial loss—and that 64% or more would be available for the direct surface irrigation of the land. Under this supposition, the corresponding depth measured at the diversion point would be (less than 1.80 ft. ÷ more than 0.64) = somewhat less than 2.81 ft. Mr. Duryea.

Although realizing that most irrigators will dissent from this conclusion, it is the writer's judgment that for such irrigation districts as are under discussion (where it is very desirable to keep the ground-water at least 6 ft. below the ground surface and usually difficult to do so on a large proportion of the district), an average irrigation depth of 2.5 ft. per year, referred to the gross area and measured at the diversion from the river, is advisable and not unreasonably low, and is sufficient to furnish adequate irrigation to lands which have been checked with reasonable skill and care, and when irrigated with reasonable skill.

The gross area of the South San Joaquin Irrigation District is 71 050 acres. In the writer's judgment, the necessary and advisable water supply is about 600 cu. ft. per sec., as measured at the diversion point from the river, or about 480 cu. ft. per sec., as measured at the edge of the District lands. Expressed in this way, the corresponding advisable "duties" of the water are about 118 and 148 acres per cu. ft. per sec., respectively.

To permit of building up reservoir storage, even while irrigating, the headworks, the canals, and the tunnels above the reservoir site were proportioned for much greater flows; and to permit of a possible "peak use" of the water and to meet desires for an ample water supply, the canal below the reservoir was proportioned to deliver 650 cu. ft. per sec. at the edge of the District, corresponding to a "duty" of about 110 acres per cu. ft. per sec. The gate outlets from the Branch Canals and District Ditches are proportioned to deliver water on the lands at a duty of 134 acres (gross area) per cu. ft. per sec.

The writer is informed by a skillful irrigator of the South San Joaquin District that (on land prepared with reasonable care by "border" or "strip" checking) he is able to irrigate alfalfa adequately on sandy land with a "head" of 16⅔ cu. ft. per sec., at the rate of 20 min. or somewhat less per acre. That head flowing for 16⅔ min. is equivalent to 4.5 in. of water on the net area.

The writer's remarks apply almost equally well to the discussion by Mr. Rudolph W. Van Norden.

Mr. Wadsworth.

H. H. WADSWORTH,* M. AM. SOC. C. E. (by letter).—This paper and the discussion on it have been read with great interest. On account of his connection with its preparation, the writer has noted with considerable satisfaction the generally expressed commendation of the report of the Advisory Board of Army Engineers to the Secretary of the Interior on the investigations relative to a water supply for San Francisco and the Bay communities.

On the appearance of Mr. Cory's paper, notes were made of several points on which the writer at first had aspirations of basing a discussion. He discovered, however, that with very few exceptions these points bore on the relative development of the Los Angeles and the San Francisco Bay regions; and it seemed wiser to him not to enter this controversy.

In considering the merits of the several distant sources of supply which have been proposed, the author mentions the necessity for caution in using the writer's figures. The writer does not question the propriety of using great caution in this respect, but suggests that a careful reading of his report will show that, as far as practicable, the unit costs applied to the City's Hetch Hetchy project were used in estimating costs of developing other proposed sources; and that successive installations were as nearly equal (in volume) for the different schemes as practicable, so that whether the estimated costs are or are not close approximations; relatively, they are so.

If, owing to the present cheapness of power derived from other sources, the value of hydro-electric power is much less than that used in the report mentioned, the present value of sources of water supply with possibilities of large power development relative to others with no such possibilities may be aggrandized. It can scarcely be doubted, however, that the ultimate (and probably not distant) value of such possible power plants will be substantially that of the capitalization of the market value of the power which they may produce.

As pointed out by Mr. Cory and also by Mr. Van Norden, the actual future need of water from the Tuolumne River for irrigation is uncertain. The writer wishes to call attention, however, to the fact that his estimate of such need is not so materially less than others, as would appear from Mr. Van Norden's discussion.

As stated in his report transmitted to the Secretary of the Interior, and as quoted in the letter of the Director of the United States Geological Survey to the Secretary of the Interior in response to the Senate resolution mentioned by Mr. Van Norden, the writer has taken the duty of water for the Turlock and Modesto Districts at 2½ acre-ft. per acre of land actually irrigated, and has allowed 23% for evaporation and seepage losses. For the total irrigation need from the

* San Francisco, Cal.

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Tuolumne River, he applies this duty to an area 60% in excess of the aggregate area of the two districts and gets an annual need of 1 132 000 acre-ft., as against a maximum requirement for any one year of 1 042 043 acre-ft. for the Turlock and Modesto Districts, as reported by Mr. Burton Smith. When volumes of water as large as either of these come to be needed, the time will have arrived for introducing such forms of construction as will decrease waste; and actual losses should be reduced to a figure not very different from the writer's assumption.

The differences pointed out by Mr. Van Norden in results reached by Dr. George Otis Smith, Chief of the Geological Survey, and by the writer, are due to differences in assignment of substantially the same total storage capacity to irrigation and to city water supply use, respectively, and to his (Dr. Smith's) allowing full estimated desirable irrigation use during the extremely dry periods like 1898-99 and 1912-13, at the expense of municipal use, rather than skimping the irrigation use (but not the acquired rights) over an area much in excess of that now in organized districts, or of that now using water for irrigation, so that a full municipal demand of 400 000 000 gal. daily may be supplied through such periods.

Mr. Hammatt.

W. C. HAMMATT,* M. AM. SOC. C. E. (by letter),—The writer is of the opinion that the attacks on the Hetch Hetchy project are a little ill-timed, particularly in view of the discussion by Mr. Van Norden. Bonds have been voted for the fulfillment of the project, part of the necessary land has been acquired, the Government permit has been obtained, a compromise agreement satisfactory to both parties has been entered into between the City and the Irrigation Districts, and ratified by Congress in the form of the Raker Bill. Actual construction has commenced on the road and railroad work, as well as on the diversion tunnel preliminary to the construction of the dam.

At this stage of the proceedings, the only thing which would justify the re-opening of the question is a decided proof of the inadequacy of the water supply, or the impracticability of its utilization. In the absence of such decided proof, the criticism becomes purely destructive, and should be condemned as such. Has any such proof been brought forth? At the present stage, comparisons of the Hetch Hetchy project with the other proposed sources of supply are beside the mark, and only the arguments against the project itself need be considered.

The Freeman report was not solely the work of one engineer, but the collaboration and compilation of many, and no expense was spared to obtain all the possible data necessary to insure the accuracy of the conclusions reached. The subsequent report of the Board of

* San Francisco, Cal.

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Army Engineers checked this report in its essential points. The only things to be considered, then, are the weight of the contradictions made by other engineers of the conclusions reached from these data.

First, in regard to the cost of the project: Mr. Van Norden has questioned the prices based on the construction cost of the Los Angeles Aqueduct. It is possible, and even probable, that his contentions are correct. It is also possible that changes in conditions of labor and materials between the time when the report was made and the time when the construction will be done will affect these prices still further. This, however, was a matter pertinent to the case of the comparison between the Hetch Hetchy and other available sources of supply, but is not a pertinent matter now that the decision has been made. If the work should cost more than the preliminary estimates (made solely for the purpose of determining the practicability of the project) showed, it will not be the first public work which has cost more than the estimate. Should the additional cost be sufficient to require the raising of additional funds, it will not be the first municipal project which has required an additional bond issue for its completion. The estimates of cost of the other projects submitted are subject to the same reservations.

Second, regarding the feasibility of the pressure tunnel, thrown into doubt by Mr. Van Norden: Apparently, he overlooked the fact that the major portion of the proposed tunnel is pressure in design only. The location of most of the tunnels is such that they carry only sufficient pressure to insure the necessary flow. In the language of the Freeman report (page 121):

"The aqueduct for substantially the entire distance down from the mountains to the edge of the San Joaquin Valley is proposed to be built of the pressure tunnel type, but with barely sufficient pressure to make sure that its section will always be filled under the various conditions of future use."

The exceptions to this are the power-drops, and although these were estimated as tunnels in the Freeman report, there is nothing to hinder their construction in the usual manner, should this be deemed advisable when the final details of the system are worked out. The power-drop tunnels are not essential parts of the project, and the details given in the preliminary report are a small matter on which to base a condemnation of the scheme.

Third, in regard to the sufficiency of the supply: One of the main points on which the opponents of the Hetch Hetchy project base their opposition is the question as to the adequacy of the flow of the Tuolumne River to furnish San Francisco with the necessary supply after deducting the needs of the irrigable lands in its catchment area. In the determination of this matter, the acreage is a known quantity, but the demand of water per acre irrigated is the point on which the

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determination of the ultimate quantity needed hinges. One reason that the water required for the irrigation of a certain crop is so difficult of determination by comparison with the use on similar lands for which data are available, is that the use in such localities is generally based on the quantity of water available rather than on the actual quantity necessary. There is no doubt that where water is plentiful an excessive use has been made by irrigators, due to their desire to get all they are entitled to, regardless of the effect it may have on their own and near-by land. It is also a fact that much water has been wasted by inefficient methods of application. The data gleaned from systems under these conditions have given rise to much of the doubt as to the adequacy of the 2½ acre-ft. per acre estimated as the ultimate irrigation need of the irrigable land under the flow of the Tuolumne River. There is, however, a constant tendency of irrigation systems, and a consequent constant decrease in the waste of water.

The writer had occasion to make very accurate determinations, as to canal losses and as to the duty of water applied to the land under the San Joaquin and Kings River Canal and Irrigation Company's system, extending over several years. The results of the investigations were as follows:

(1).—That 45% of the water taken into the system was lost by percolation and evaporation in the entire system.

(2).—That the quantity of water measured as flowing to the land for irrigation was 2.44 acre-ft. per acre for 1907, 1.49 acre-ft. for 1908, and 1.44 acre-ft. for 1909.

The drop in the quantity used in 1908 below that used in 1907 was due to the company's change in irrigation charge from an acreage basis to a quantity basis. Prior to 1908, the rate was made per acre irrigated, regardless of the quantity of water used thereon, but, in that year, the rates were changed to so much per second-foot for 24 hours. This change was made for two reasons: First, because the extraordinary demand for water was forcing the system beyond its capacity; and second, because the excessive use was drowning the land. The result of the change was fully up to expectations, as the irrigators, when they were obliged to pay for the water used, reduced their use to their needs. Although the company has since returned to the acreage basis of charges, the educational work has been lasting, and the demand still remains as it was during the years when the rates were fixed on the quantity basis. The duty previously given was for all crops, the prevailing crop, however, being alfalfa, about 70% of the land being irrigated for this crop.

The writer is very familiar with the lands on the east side of the San Joaquin Valley from the Stanislaus River to Kings River, and

Mr. Hammatt. has made several examinations of irrigation systems in this area. From his knowledge of the conditions in this territory, he is able to deduce the following conclusions:

(1).—That the canal losses in the Turlock and Modesto Irrigation Districts would be considerably less than those under the San Joaquin and Kings River Canal and Irrigation Company's canal system, due to the lesser canal length and greater fall of the former. The losses from seepage and evaporation in the canals of the Modesto and Turlock Irrigation Districts would undoubtedly fall below 35% of the total intake.

(2).—That the necessary use of water on the land in these districts would be at least as small as that under the San Joaquin and Kings River Canal and Irrigation Company's system, as the soil evaporation would be less and the sub-surface drainage probably no greater. This conclusion is deduced from an intimate knowledge of the soil conditions in both localities.

The necessary use of water, then, as measured at the intake, would be, considering 1.5 acre-ft. per acre necessary on the land itself, $1.5 \div 0.65 = 2.31$ acre-ft. per acre. Or, if the gross acreage is considered, as about 20% of the land is occupied by roads, buildings, yards, and other non-irrigated tracts, the necessary quantity for the whole area would be 80% of 2.31, or 1.85 acre-ft. per acre. It would seem, therefore, that the allowance of 2.5 acre-ft. per acre referred to in the other discussions would be ample to meet all contingencies of waste due to canal regulation, individual excessive use, and whatever other extraordinary condition might arise. Although there is little question that a slightly greater quantity than this has been used in the past on these lands, it is apparent that the quantity used has been in excess of the needs, as is shown by the rise of groundwater and the drowning of the crops in many localities.

Mr. Griffin. A. GRIFFIN,* ASSOC. M. AM. SOC. C. E. (by letter).—One who has followed the efforts of San Francisco's representatives to secure the Hetch Hetchy water supply for San Francisco, and who has read the papers and talked with the residents of San Francisco, cannot have failed to observe the lack of a grasp of the true situation. This observation applies with almost equal force to the engineer and to the layman, and the silence of the San Francisco press on every subject in the least degree unfavorable to the Hetch Hetchy project is very surprising.

It would seem that, in planning a great municipal water supply, at a cost of $50 000 000, the adequacy of the source should be ample, even to excess, and that the engineering and legal advisers of the project should report adversely unless the source is shown to be ample under the most unfavorable assumptions. Yet we have the peculiar

* Manteca, Cal.

spectacle of the Hetch Hetchy burden being laid on the taxpayers of San Francisco, supported by unsafe assumptions, by half statements of facts, and by absolute silence on all unfavorable features, when it is known, beyond the possibility of a doubt, that the works cannot be supplied to capacity during critical periods which recur with moderate frequency. This is made known even by computations based on assumptions as to storage capacity and irrigation use which are calculated to show the greatest possible quantity available for a municipal supply. Mr. Griffin.

There is actually no definite and conclusive knowledge as to the quantity of water land requires, nor what represents the most efficient use, nor what represents the greatest beneficial use to any person or community, which last is the quantity for which the Modesto and Turlock Districts can rightfully contend. These quantities, in fact, are variable from year to year, depending on the crops, rainfall, prices, and many other factors. The computations which show 400 000 000 gal. daily available for San Francisco are based on an assumption as to the annual use in irrigation, which might easily be 50% greater, and, if it were 20% greater, the quantity shown by the computations would be wholly inadequate, not to mention the fact that, if the assumptions as to the monthly use of water and the quantity of storage available had been more conservative and more in accord with the probable facts, the quantity shown as available for San Francisco would have been decreased still further.

The danger of assuming too small a water requirement for land is illustrated forcibly by the history of the Reclamation Service. The original assumptions led to the making of contracts with settlers to deliver from 1½ to 3 acre-ft. per acre per annum to the land. In many instances it has been found necessary to modify these contracts, if not legally, by common consent; and on some of the projects the United States Reclamation Service is actually delivering 10 and even 20 acre-ft. per acre per annum to some of the lands, and the officials in charge of operation consider these quantities necessary for the best agricultural results. Although extreme conditions are represented by such uses, the general testimony of officials of the Reclamation Service is that it was necessary to revise the first assumptions as to the water requirement of irrigated lands in favor of the use of a greater quantity.

A feature which has been brought forcibly to the writer's attention at various times while engaged in irrigation operations is that the years of deficient run-off are accompanied by deficient winter and spring rainfall in the adjoining irrigated areas; therefore, there is an earlier, a longer, and a greater demand for water for irrigation; that the total quantity used is increased over that used under normal conditions; and that the seasonal distribution of this use is altered.

Mr. Griffin.

It has also been assumed that the Modesto and Turlock Irrigation Districts will develop foot-hill storage to the maximum capacity of the basins tributary to their canals, though, as a matter of fact, it is doubtful whether it will ever be economically feasible to develop them to much more than half this maximum, due to the increasing values of the sites for agricultural purposes and the uncertainty of filling the basins through their two diverting canals.

Aside from the probability of the actual failure of San Francisco to get the desired supply from the Tuolumne, the consequent loss from lack of water, and a partial use, only, of an enormous investment, and the possibility of having to duplicate the investment to secure an additional supply from another source, there should be considered the case of the lands outside the Modesto and Turlock Districts which are dependent on the Tuolumne for a water supply. According to the report on the Irrigation Resources of California, by Mr. Frank Adams,* there are 450 000 acres of valley and plains land tributary to the Tuolumne, and an additional area of foot-hill land between the Stanislaus and the Merced Rivers of 180 000 acres, of which it may be assumed that one-half, or 90 000 acres, is tributary to the Tuolumne, making a total of 540 000 acres tributary to that stream. From personal familiarity with these lands the writer knows that practically all of them are susceptible of being irrigated beneficially; but it is a fact that economic conditions do not warrant the expenditure of the money necessary to provide them with a water supply to-day, and may not warrant this expenditure for years to come. The possibilities of development are there, however, and can never be realized until water is applied to them; and, if the Hetch Hetchy project is carried to a conclusion, most of them can never receive water. This is rendered the more certain on account of the proposed development of so many of the reservoir sites of the Tuolumne for municipal supply. The lands yet to be supplied with water will have inferior water rights, and will be much more dependent on storage than the Modesto and Turlock Districts. The control of 1 acre-ft. of stored water for late irrigation would make possible the use of from 1 to 3 acre-ft. of natural flow for irrigation of the same land in the spring and early summer; but, with the proposed depletion of the total supply of water in the river and the utilization of the best reservoir sites for the San Francisco supply, there would only be left the inferior and expensive reservoir sites and the peaks of the floods for the latter lands, with the probability of no water supply at all in critical periods. This would make any development beyond that at present inaugurated impossible, from an economic standpoint.

* Issued as a State Report in 1912, and also issued by the Department of Agriculture as U. S. *Bulletin 254*.

Mr. Griffin.

The report mentioned previously shows that the San Joaquin Valley has an area of irrigable or agricultural land of more than 8 000 000 acres, with a mean annual run-off of about 11 000 000 acre-ft. and a minimum run-off of about 5 000 000 acre-ft., and that the Sacramento Valley has a similar area of somewhat more than 4 000 000 acres with a mean annual run-off of 26 000 000 acre-ft. and a minimum run-off exceeding 12 000 000 acre-ft., and with a much higher annual precipitation over the agricultural lands. One cannot help being impressed with these figures, and, knowing that water can be taken to San Francisco from the Sacramento Valley at a cost not greatly different from that of the Hetch Hetchy project, with an absolute assurance of its adequacy, one wonders at the persistent partisanship of the people who have led San Francisco into the Hetch Hetchy scheme.

It would seem that, if only for the selfish reason of permitting a tributary community to develop to its highest in order to increase its buying and selling capacity, San Francisco might well keep out of the San Joaquin Valley. Aside from the actual loss that will accrue through arrested development, there is a growing personal antagonism to San Francisco on the part of many people of the San Joaquin Valley, and this has already diverted trade from San Francisco to other centers; and, as time goes on and it is seen more clearly how this enterprise has stifled development, and when, during critical periods, San Francisco shall become involved in altercations and legal controversies with the irrigation interests over the division of water, this feeling will be greatly intensified. Even to-day the Modesto and Turlock Districts, which can be injured very little if the terms of the Raker bill are adhered to, have filed suits against San Francisco, and from time to time are making strategic legal moves for the purpose of checkmating her wherever possible.

Mr. McFarland.

J. Horace McFarland,* Esq. (by letter).—After several years of more or less close connection with the so-called Hetch Hetchy project for the water supply of San Francisco and its appurtenant communities, the writer has a natural interest in this paper.

The position of the American Civic Association, in this matter, is as follows: Those who make up that Association are a unit in the position that no use of water is so important and so beneficial as that for domestic purposes, and at no time in the Hetch Hetchy controversy have they lost sight of this basic fact.

They have felt, however, all the time, that before consenting to the virtual destruction and withdrawal from a possible very beneficent use of a unique and unreplaceable portion of the world's surface, absolute necessity should be shown.

* President, American Civic Association, Harrisburg, Pa.

Mr. McFarland.

Before committees of Congress, before successive Secretaries of the Interior, and, at various times, the writer has contended, first, for the absolute necessity of the best possible domestic water supply for any city; and, second, for that fair and impartial investigation as to various sources of supply which he is constrained to believe, even now, after following the matter for at least six years, has never yet been made in relation to San Francisco.

Having heard voluminous statements by many engineers, and having read countless pages on the subject, he is still of the opinion that the basis of the engineering investigation with respect to the water supply of San Francisco has never been impartial. All the engineers employed by San Francisco and all those called to support their findings have devoted themselves with one voice to exploiting the Hetch Hetchy as the only supply, and to minifying, instead of investigating, other possible sources.

Now, given equal purity and potability, a glass of water from one source is as useful to the residents of any city as a glass of water from another source. Taking into account the turning of the American people toward scenic phenomena and the beginning of a recognition of the magnificent scenery in America to attract visitors, it is, and has been, fair to assume that any great scenic source of water supply ought to be very thoroughly studied, not with reference to its commercial availability for power and then its incidental availability for a domestic supply, but from the standpoint of being used only as a last resource, after all other economically possible sources have been considered.

The writer does not believe this has ever been done, and, as respecting the findings that have been recorded so far in regard to the Hetch Hetchy supply, he is constrained to repeat a remark made in his hearing by a very vigorous ex-President of the United States, referring to a certain Goverment official who was just then terminating a career of actually pernicious activity. He said: "If Mr. B. believes a man to be guilty, he sets out to find evidence to back up his belief. If he doesn't find it, Heaven help the evidence!"

At least three mayors of San Francisco and two succeeding city engineers, together with a dozen other high officials of that enterprising and wonderful city, would testify in the writer's favor that at every hearing where they have met he has reiterated the statement that if the Hetch Hetchy Valley was shown to be the only practicable source of a satisfactory water supply for the City of San Francisco, the American Civic Association would be the very first organization to recommend that it be given up to that city; but San Francisco has never shown this.

The extreme value in promoting and maintaining human efficiency by great recreation places, open to all the people; the great economic

value of unmatched and unique scenery to attract travel and, therefore, revenue to the United States from all the world; a decent respect for the unique glories of our own land, and a regard for the findings and opinions of a man so exalted in his attitude and so high in his knowledge as the late John Muir—all unite in promoting the feeling that the present situation of the Hetch Hetchy matter in its relation to the City of San Francisco is working out to a great wrong to that city, to the United States, and to the whole world. Mr. McFarland.

F. C. HERRMANN,* M. AM. SOC. C. E. (by letter).—The author has taken a very large subject, and, covering it in a broad and general way, has omitted the many interesting engineering features relating to the present water supply system of San Francisco. It is to be regretted that members who have been connected most intimately with the problems of this work and their solutions have not participated in the discussion. Mr. Herrmann.

By reason of climatic and topographic conditions in the region of San Francisco, the problems were unique; and, because that city was isolated from other large centers of engineering activity during its development, their solution was independent in character, often resulting in bold engineering, such as the early use of long lines of thin, riveted wrought-iron pipe of large diameter, high earthen and concrete dams, and ashlar masonry dams of the arch and buttress type with slender cross-section.

Early in the history of these works it was found that long cycles of years having less than average run-off were likely to occur, and, to provide against them, copious storage was essential. The rule was formulated that there should always be available storage capacity equivalent at least to the consumption for 1 000 days.

It is to be hoped that the engineering accomplishments of this system will be presented first-hand to the Profession before very long.

In his discussion, Mr. O'Shaughnessy makes unfavorable comment on the character and reliability of the run-off records of the Spring Valley Water Company, as well as the estimates of water crop by a number of engineers, including the writer. It is to be presumed that he means the Alameda system only, for the records of the peninsular system are probably the best and most reliable anywhere in the western part of the United States.

Water-works engineers of wide experience who have had occasion to examine carefully the hydrographic records of the Spring Valley Water Company have stated that they are very much better than is usual with water-works systems. From his own experience, the writer knows that they are far superior to those existing generally in the West and Middle West.

* San Francisco, Cal.

Mr. Herrmann.

The Alameda system is as yet but slightly developed. There remain to be constructed three reservoirs having an aggregate storage capacity of more than 77 000 000 000 gal., one of which is the Calaveras Reservoir (52 000 000 000 gal.) which is now being built and will be used during the coming season. A number of engineers have made careful estimates of the yield of the Alameda system; others have made statements of yield without careful analysis. The results differ widely, just as they have differed widely in the case of the Tuolumne River. The wide variation in the estimates of yield of the Alameda system is due in some degree to the use of different fundamental elements, such as storage available and allowances for future local consumption.

The waste water of the Alameda system (650 sq. miles above Sunol Dam) finds its way into San Francisco Bay through Alameda Creek. Record has been kept of this by measuring the depth of water over the Niles Dam from 1889 to 1900, and over the Sunol Dam from 1900 to date. In times of high flood both these dams act as submerged weirs, with high velocities of approach. The discharge was computed from these depths by the Spring Valley Water Company by using the formulas, $Q = 4\ 200\ b\ h^{\frac{3}{2}}$ for the Niles Dam, and $Q = 4\ 400\ b\ h^{\frac{3}{2}}$ for the Sunol Dam, which was adopted by Hermann F. A. Schussler, M. Am. Soc. C. E., after numerous observations and measurements. Mr. Schussler has always maintained that discharges computed by these formulas were conservative, particularly in stages of high flow.

Unfortunately, the United States Geological Survey published a record of the flow of Alameda Creek over the Niles Dam, which was fragmentary and based on wholly erroneous data, but indicated that this flow was very materially less than is shown by the records of the Spring Valley Water Company.

This was made quite an issue in the investigations of 1912 by the City of San Francisco preparatory to the hearing before Secretary of the Interior Fisher. Those in charge of the City's affairs appointed a committee composed of C. D. Marx, President, Am. Soc. C. E., of Stanford University, Charles Gilman Hyde, M. Am. Soc. C. E., of the University of California, and C. E. Grunsky, M. Am. Soc. C. E., former City Engineer of San Francisco. The duties of this committee were to make an analysis of the discharge of Alameda Creek over the Niles and Sunol Dams, and of Calaveras Creek at the Calaveras Dam site, from the original records of the Company.

These gentlemen, with the necessary assistants, made a very careful and exhaustive examination of the records, and a survey of the stream bed and structures.

Mr. Herrmann.

The controversy as to the flow of Alameda Creek arose during the period of low-water flow, making it impossible to calibrate the dams by stream measurements prior to the hearing before Secretary Fisher. Messrs. Marx, Hyde, and Grunsky, therefore, made discharge curves for the dams from theoretical analysis.

At the time that these gentlemen began their investigation, Professor J. N. LeConte, of the University of California, conducted a series of experiments for the writer with models of these dams. These models and the approaches to them were exact miniature counterparts of the structures and the stream bed above them as they existed during the times of stream flow observations. Their dimensions were one-twentieth and one-nineteenth of those of Sunol and Niles Dams, respectively. The models were by no means small, having crest lengths of more than 6 ft., and they were calibrated for different rates of flow, from about $\frac{1}{2}$ to 17 sec-ft., and for different degrees of submergence.

Diagrams* were prepared showing the relation of up-stream head to submergence for various values of Q, the discharge curves for the models for different degrees of submergence, the water-surface line at the two dams for the flood of March, 1911, and the arrangement of the experiment station. There was also a diagram of the models.

The flow of water was measured through sharp-edged weirs, using the standard formula, $Q = C \frac{2}{3} \sqrt{2g}\, b\, h^{\frac{3}{2}}$, the coefficient, C, being taken from tables for contracted weirs by the late Hamilton Smith, M. Am. Soc. C. E.

The heads were measured with hook-gauges reading to 0.001 ft., and the degree of submergence was controlled by baffles placed 30 ft. below the models. The conditions were ideal to secure a constant flow for a full set of experiments. The experiments, and the application of their results to the large dams, have been described by Professor LeConte in his report to the writer.

His results were made available to Messrs. Marx, Hyde, and Grunsky before they had finished their analysis. His work, as well as that of Messrs. Marx, Hyde, and Grunsky, was very severely and unjustly criticized by those representing the City of San Francisco at the hearing before Secretary Fisher.

During the winters of 1912-13 and 1913-14 the Sunol Dam was calibrated by careful stream gaugings taken just above it. Independent measurements were taken by the engineers of the City of San Francisco and the engineers of the Spring Valley Water Company; the data taken by both agreed remarkably well up to a gauge height of 8 ft. For gauge heights above that point complete current-

* These diagrams are not reproduced herein, but are filed in the Library of the Society where they may be examined by those who are interested.

Mr. Herrmann.

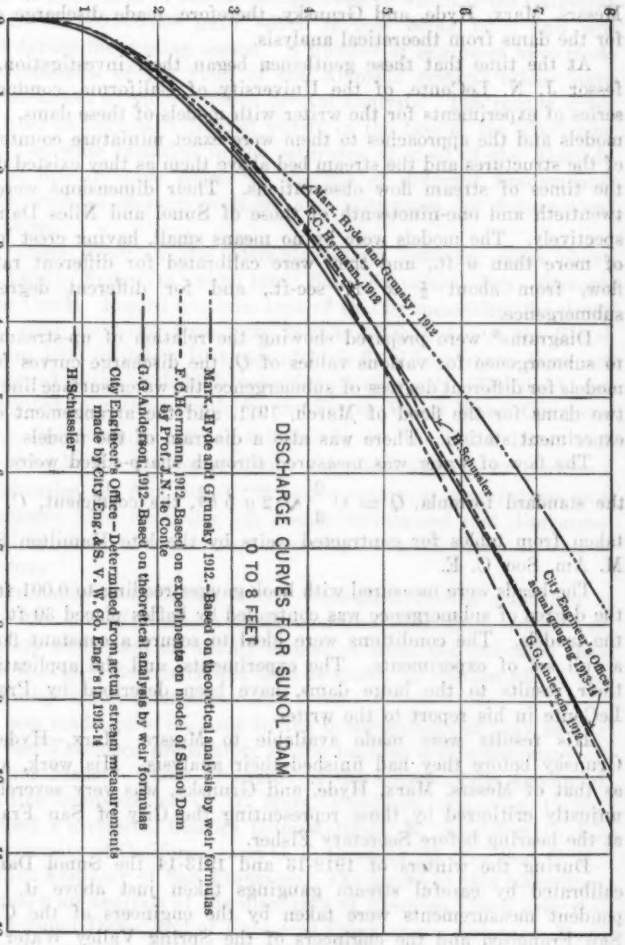

FIG. 28.

meter measurements were impossible because of the high velocity of the water, and satisfactory data were not obtained. However, as the great bulk of the run-off occurs for gauge heights of less than 8 ft., all the current-meter measurements were assembled, and a discharge curve for the Sunol Dam was constructed by the City Engineer of San Francisco. This curve, together with those obtained from the LeConte experiments, the original curve prepared by Mr. Schussler, the Marx, Hyde, and Grunsky computations, and computations by G. G. Anderson, M. Am. Soc. C. E., is shown by Fig. 28. Inspection of these shows the very close agreement between the results obtained by the model of the Sunol Dam and by actual stream measurements. Computation of the total flow of Alameda Creek over Sunol Dam from 1900 to 1913, by the curve from actual gaugings, is about 3% in excess of that obtained by the curve constructed from the LeConte experiments on the model of that dam. Mr. Herrmann.

Much has been written about the use of water for irrigation from the Tuolumne River by the Turlock-Modesto Irrigation Districts, and computations of their future requirements made by different engineers vary between very wide limits. The Modesto Irrigation District feels that it has need for more water than is provided for under the Raker bill, and has brought suit against the City of San Francisco in the California Courts to adjudicate these rights. At the present time it is spending upwards of one-half million dollars to increase the capacity of its main canal, in order that it may divert 2 000 sec-ft. from the Tuolumne River at LaGrange, and is seriously considering the development of large storage at much greater cost, so that it may prolong the irrigation period, and be secure against the critical years of small run-off.

WALTER HENRY BROWN,* M. AM. SOC. C. E. (by letter).—In this paper, as well as in the many discussions brought out by Mr. Cory's treatment of the problem of the water supply of the so-called, but yet unorganized, San Francisco-Oakland Metropolitan District, there is an evident absence of any definite suggestion as to how the Metropolitan District may best secure an enlarged and adequate water supply from any source other than the Hetch Hetchy region. It is clearly the view of many who have participated in the discussions that the Hetch Hetchy project does not offer the economical possibilities desired, yet the contributions are lacking in any enlightening statements as to where the needed water may be obtained—elsewhere than from the Hetch Hetchy source. Other projects have been suggested, in a tentative way, but Mr. Cory has taken occasion to unfold to view what he considers weak points in the Hetch Hetchy scheme, otherwise known as the Freeman Project, but he does not offer a Mr. Brown.

* San Francisco, Cal.

Mr. Brown.

solution for the difficulties—the impossibilities, if you will—he has discovered.

Mr. Cory has always been painstaking in his professional work, and unusually thorough; and it is to be regretted that in this paper, the result of extensive research and experience in and with the local water problem, he should have criticized, severely yet perhaps with justification, the plans proposed in the Freeman Project, without offering a feasible plan for meeting the water requirements of the cities on the Bay of San Francisco.

Mr. Rhodin mentions the feasibility of a filtered supply from the San Joaquin or the Sacramento Rivers, also direct draft from the McCloud River, Indian Valley, and Yuba River, as offering possible solutions. Eel River, in the Coast Range Mountains, to the north of San Francisco, is mentioned as an opportunity possessing considerable merit, but no suggestion of the probable cost is offered.

Mr. Grunsky refers to the Eel River source in the following sentence:

"The water supply problems to the northward of San Francisco Bay are separate and distinct from those of San Francisco and the East Bay cities, and need not be brought under discussion in connection with the larger San Francisco problem."

The question arises in the writer's mind: Why need it not be brought under consideration and discussion in connection with the larger San Francisco problem? If searched for, there might be reasons for the acceptance of the Eel River source not yet brought to the attention of Mr. Grunsky, or of Mr. Cory who has essayed to expand on the Greater San Francisco water problem, criticizing the only source seriously considered, yet neglecting to suggest an alternative.

As a participant in the discussion of this paper, the writer cannot expect, at this date, to go into the engineering and cost details of the Eel River possibilities; but, being familiar with that region for the past seven years, attention is called to the chapter of Mr. Freeman's Report which is devoted to that phase of the situation.

This chapter frankly admits that no study of the Eel River possibilities has been made, and is warrant, in the writer's belief, for some earnest effort being made toward determining the value of the Eel River region as a source of supply for municipal needs. In the first place, it is not necessary, or even desirable, to make the bay crossing under the Golden Gate in the presence of the more favorable crossing offered at McNear Point, near the City of Richmond. San Francisco is not, of necessity, the only objective point in the proposed Metropolitan Water District. Oakland, Alameda, Berkeley, Piedmont, Richmond, and many other thriving cities are expected to contribute toward the construction costs of a new and great water

supply, and their interests should be considered in the selection of, as well as in the payments for, a new supply; and, for these combined communities, the Golden Gate route has no part in the problem, under any form of development of the Eel River sources as yet in evidence. Mr. Brown.

Cost of Initial Development of Eel River Sources.—Construction estimates have been made for an initial development of the Eel River supply for from 12 000 000 to 15 000 000 gal. daily. Peculiar as it may seem, it would not be profitable to bring into the limits of the proposed Municipal Water District, at this time, more than 15 000 000 gal. daily, at the outside. The writer was very much surprised when this state of affairs was developed through the analysis of evidence given at a recent hearing before the California Railroad Commission in connection with the East Bay water situation. The reason there is not, at this time, justification for bringing in more than 15 000 000 gal. daily is that the Calaveras Dam is well along under construction, and will provide for all needs of San Francisco proper for the generation now interested in these affairs. The present consumption of the East Bay Cities is somewhat less than 18 000 000 gal. daily, and, with the existing local resources, there is not room for the disposal of more than 15 000 000 gal. daily, which might be secured from a distant source.

Of course, the cost of water per 1 000 gal. to the consumer diminishes with an increased capacity of the supply system, but even so small a quantity as indicated can be delivered into the local storage of the East Bay Cities, at an elevation of 200 ft. above sea level, for less than $5 000 000, gross construction expense, with attending gross operating and overhead charges of less than $1 500 per day. These figures cover the cost of building a pipe line from Eel River, with a capacity of 20 000 000 gal. daily, and would enable the delivery of water to the Bay District for a rate of less than 15 cents per 1 000 gal. The Eel River sources, without storage at points of diversion, by conduit route 130 miles from San Francisco, and utilizing less than the existing terminal storage facilities, will, without question, supply 12 000 000 gal. daily, and, with the storage facilities available at the head-works, at Gravelly Valley, 180 000 000 gal. daily may be depended on as the ultimate safe yield from that particular unit of the Eel River development. When more than 180 000 000 gal. daily are required for the Metropolitan District, there is more water available in the Middle Fork of Eel River, 38 miles north of the South Fork, where an additional supply of 350 000 000 gal. daily may be obtained. This gives a total of 530 000 000 gal. daily from Eel River.

The writer has made extensive studies of the Eel River regions, in connection with work other than the Greater San Francisco water problem, and offers to his associates in the Profession, who are in-

Mr. Brown. terested in this particular problem, the following notes, subject to further expansion should interest warrant:

1.—The combined Eel River sources are suitable for a programme of gradual and economical development without throwing excessive or undue burdens on any one generation of the communities' inhabitants.

2.—The Eel River sources as here proposed have never, at any time, been investigated by the City authorities, or by the Advisory Board of Army Engineers, as a source of municipal water supply.

3.—By taking a municipal water supply from the Eel River sources, no community interest can, in any way, be threatened or jeopardized by being deprived of water. Litigation, such as threatened in the San Joaquin Valley over established and valuable water rights in connection with the proposed Hetch Hetchy diversion to San Francisco, would thus be obviated.

4.—Irrigable lands in the valley of the Russian River, now without irrigation, could be supplied with excess water from the Eel River sources, following the example or policy of Los Angeles in building up and making more productive its adjoining areas.

5.—To leave to the San Joaquin Valley the water available to it, even though the farmer's ideas of his needs may appear extravagant to the man in city life, will result in that community's growth in population and commerce, and actual thrift, which will naturally center in and build up the cities on the Bay of San Francisco.

6.—The region of the Eel River water-shed, above points of diversion, is in its virgin state. Its mountain ranges run from 6 000 to 8 000 ft. above sea. The water therefrom runs free of pollution by habitations; the maximum hardness at periods of lowest water is 17°, French scale; there are no lumber camps or summer resorts on its areas; and the entire water-sheds of both branches of Eel River are enveloped by the Stony Creek National Forest.

7.—Precipitation on large portions of the Eel River region occurs at the rate of 100 in. and more per annum. There are no requirements for irrigation, and all the water in question runs to waste to the sea.

8.—The combination of sources from the Middle Fork of Eel River and the South Fork of the same stream makes a feasible, practicable, commercial project that should receive, at least, the attention of those seriously concerned in the San Francisco-Oakland Municipal Water District problems.

Mr. Cory. H. T. Cory,* M. Am. Soc. C. E. (by letter).—The writer's purpose in preparing the paper was, first, to acquaint the members of the Society with the general local water situation, and second, to present the subject for discussion by technically trained men before a technically trained audience.

* San Francisco, Cal.

Mr. Cory.

The writer, therefore, is not only much gratified with the interest shown by the volume of the resulting discussion, but also that those who have participated include the present City Engineer of San Francisco; the ex-City Engineer who originally suggested the Hetch Hetchy source of supply and outlined the first Tuolumne project; two of the three engineers who have been called into consultation with respect to power features of the present Hetch Hetchy project, and four others who have been employed by the City during the preparation of the Freeman Report and the water-rate cases now pending on the San Francisco side of the Bay; the engineer in charge of preparing the Army Board Report; a member of the Commission which reported to Secretary Ballinger in 1910 on the need of the Hetch Hetchy Valley; the Chief Engineers of the Modesto, the Oakdale, and the South San Joaquin Irrigation Districts; and the President of the American Civic Association.

Analysis of the various comments shows that no one has criticized the writer's conclusions regarding the following:

1.—The future population curves of the San Francisco-Oakland Metropolitan District as a whole and of the several individual sections thereof;
2.—The future water requirement curves of the district and of the several different sections comprising it;
3.—The relative bonded debt, municipal construction needs, and amount of taxable property in San Francisco and Los Angeles now and in the future;
4.—The fact that the problem of a distant water supply is far more real to the East Bay Cities than to San Francisco;
5.—The fact that the East Bay Cities, particularly, should endeavor to get water costing the consumer little if anything in excess of 15 cents per 1 000 gal., and regard such cost limit as of prime importance;
6.—The essential factors affecting the selection of a distant water supply source;
7.—The advantages and disadvantages of various distant water supply possibilities—with the exception of the Hetch Hetchy and Eel River;
8.—The fact that the Sacramento Valley has an excess of water and could easily spare all of San Francisco's needs both present and future; and that the San Joaquin Valley has a deficiency of water for its own needs, no matter to what extent conserved;
9.—The desirability of a water board to handle all water supply problems, which board should be entirely separate and distinct from any and all other municipal authorities.

Mr. Cory.

It is realized, of course, that lack of criticism does not necessarily imply universal approval.

With the single exception of the present City Engineer, Mr. O'Shaughnessy, there have been no expressions of approval concerning the wisdom of San Francisco in building the Hetch Hetchy project, as determined by the provisions of the Raker Bill. Mr. Galloway, states that he is, and has been since the Grunsky proposal of the Tuolumne source in 1902, an advocate of Sierra water, and Mr. Hammatt feels that the City has gone so far in the matter that discussions of the Hetch Hetchy project are out of place. All other comments which touch on this phase of the subject are either condemnatory or doubtful.

The City Engineer alone takes issue with the statements made concerning the yield of the present water sources when fully developed.

Considerable diversity of opinion was expressed concerning the water needs of 300 000 acres of land in the Turlock and Modesto Irrigation Districts.

The discussion has been so voluminous that it is impracticable to summarize the views of those taking part, or, generally speaking, even comment on the various points brought out. As a matter of fact, there is no reason to do so, for, by a careful reading of all the discussions, one can very easily decide the proper weight to be given the various expressions of opinion.

An exception is the discussion by Mr. O'Shaughnessy, which requires some comment. He points out that the writer was in error in stating that San Francisco had more than 2 200 saloons, and that the official figures are 2 051. The writer is glad to be corrected, and will only say in explanation that the figure used, which was taken from the Assessor's records, through misinterpretation of the data, included more than 200 one-day licenses. Neither of these figures quite tells the story, because of the "corner grocery", as the U. S. Revenue Retailers licenses issued during the quarter ending June 30th, 1914, were 5 818, or about 1 in 85 of population.

He also notes that the population curves in Fig. 2 are not identical with those given out by the Census Bureau for the respective metropolitan districts. In the paper the writer was careful to state that "Greater San Francisco-Oakland" and "Greater Los Angeles", as used throughout the paper, comprise, respectively, all the territory which should logically join in a common water supply within circles of 30 miles radius. The populations in these districts are naturally not identical with those found by the Census Bureau for districts with different boundaries, although, as a matter of fact, the differences are quite small.

He also corrects the writer's figure, "about 100 miles of steel pipe", in the Freeman-Hetch Hetchy project, and states that there are 87

miles. The writer was under the impression that more than 13 miles of the pipe line was a double line, but is glad to be put aright. Mr. Cory.

Regarding the planning and construction of the Municipal Street Railway system and other City work by Mr. O'Shaughnessy, the writer is very glad of this opportunity to express his unqualified admiration therefor. All of such work has been planned along broad lines, of high character, aggressively prosecuted, and carried through without the slightest hint of favoritism or political meddling.

In spite of the writer's statement relative to Table 1, showing the estimated yield of various Spring Valley water sources, Mr. O'Shaughnessy comments as though the paper had not emphasized sufficiently the differences of opinion of the various engineers who have investigated the matter. Mr. O'Shaughnessy has stated:

"For years there has been much investigation as to both the ultimate and the available yield of the Spring Valley sources. Many reports and estimates have been filed by various parties—some representing San Francisco or the East Bay Cities, or both; others working in the interests of one or more of the water companies supplying those cities. In matters of this sort it usually happens that investigators of the latter class are very optimistic in their estimates, and those of the former class are more conservative."

A very important method of examining and comparing safe water yields with given storage capacities has been presented in a paper* by Allen Hazen, M. Am. Soc. C. E. In that paper, Figs. 40 and 41 give the curves for California conditions for 95% and 98% dry years, respectively. Throughout much of California the evaporation is in excess of the rainfall, and in the southern part of the State, particularly, the extent of over-year storage required is extraordinarily large on account of the run-off conditions shown in Fig. 17. From the safe yield, read off from Mr. Hazen's Figs. 40 and 41, excess of evaporation losses must be deducted—treated as a part of the draft. The correction ranges from nothing in the Sierras and Upper Coast Ranges to 5% in the Bay region and 25%† in the vicinity of San Diego.

Mr. Hazen's method makes very simple the estimation of safe yield throughout the various parts of the State, and is on the same basis with respect to severity of the drought conditions provided against. That is to say, years are considered as types—no one actual year being taken—and are designated or defined, so far as degree of dryness is concerned, by their position in order of dryness. For example, a year of such a degree of dryness that 95% of the years are wetter—have larger quantities of run-off—and 5% are dryer, is called a 95% dry year; similarly, a year of which it is true that 98% of the years are wetter and 2% are dryer, is called a 98% dry year.

* *Transactions*, Vol. LXXVII (1914), pp. 1539 to 1669.

† Based on 52 in. mean annual evaporation from water surfaces. The California Railroad Commission uses a larger figure.

Mr. Cory.

Thus, a 95% dry year standard means that there will be a water shortage in 1 year in 20, and, with a 98% dry year standard, shortage will occur only in 1 year in 50. Table 9 shows the length of the run-off record, mean run-off, in acre-feet, coefficient of variation, and available storage, in acre-feet, of several streams, including the important ones mentioned in the paper and in the discussions. These data were used in connection with Mr. Hazen's Figs. 40 and 41, the corrections for excess of evaporation have been applied, and the resulting net yield is given in Table 10 under the headings "95% dry year" and "98% dry year".

In 1907, Mr. O'Shaughnessy became Chief Engineer of the Southern California Mountain Water Company, supplying water to the City of San Diego. On June 5th, 1912, the City bought the property, and the bond election was held on August 15th, 1912. The vote was 6 948 to 1 405, due to assurances from the company that the plant output was a "sufficient supply of water to take care of the increased population and growth which the Panama Canal and other factors are bound to create."* The mean daily consumption in 1912 was 5 819 214 gal., and in 1913, 6 850 000 gal., the city's growth being rapid.

The system then included the Otay and Morena supplies, which were then held by Mr. O'Shaughnessy as being good for 13 500 000 gal. daily. In April, 1914, at a hearing before the California Railroad Commission in connection with related matters, Mr. O'Shaughnessy testified that he had in the past year and a half reduced his estimate of the safe yield from Morena Reservoir to 6 500 000 gal. daily, the safe yield from Otay remaining at 400 miners' inches, equivalent to 5 160 000 gal. daily. He also testified that in 1912 he had considered Cottonwood Creek, fully developed, to be good for 1 500 miners' inches (however, presumably with 46 000 acre-ft. of storage at the Barrett Dam), but now estimated that with the creation of 27 675 acre-ft. of storage at the Barrett Dam site, and the 46 000 acre-ft. of storage existing at Morena, the safe yield would be 15 000 000 gal. daily; and that the present safe yield could be increased by 5 000 000 gal. daily by creating 27 675 acre-ft. of storage at Barrett.† The Engineering Department of the Railroad Commission at the same hearing determined the safe yield of the Otay as 2 500 000 gal. daily; of the Morena, 3 500 000 gal. daily; from the Cottonwood, complete development, 5 400 000 gal. daily; and the increase in safe yield by the construction of the Barrett Reservoir, 1 900 000 gal. daily.

Within less than a year, San Diego realized that the water supply must be increased, and Mr. O'Shaughnessy and J. B. Lippincott, M. Am. Soc. C. E., were called in to report on additional proposed sources

* "Construction of the Morena Rock Fill Dam, San Diego County, California," by M. M. O'Shaughnessy, M. Am. Soc. C. E., *Transactions*, Vol. LXXV (1912), p. 51.

† Transcript of Evidence, Application No. 547, California Railroad Commission, pp. 637 ff and 675, and Exhibit 10.

Mr. Cory.

TABLE 9.

Stream.	Record.	Mean run-off, in acre-feet.	Coefficient of variation.	Available storage, in acre-feet.
Tuolumne River	19 seasons	2 095 000	0.410	1 000 000 1 500 000 1 800 000
San Leandro Creek	36 seasons	18 164	0.676	14 858
Eel River	9 seasons	552 000	0.473	215 000
Temescal Creek	9 seasons	1 162	0.663	460
Calaveras Creek	16 seasons	50 400	0.605	162 700
Upper Alameda		15 700		
Total		66 100		
Crystal Springs System	25 seasons	20 700*	0.610	92 070
Boulder Creek	26 seasons	4 550	0.721	11 000
Lake Cuyamaca				
San Luis Rey River at Warner's	18 seasons	20 920	1.011	162 000
Cottonwood Creek at Barrett	9 seasons	20 479	0.880	73 675
Cottonwood Creek at Morena Dam	7 seasons	8 150	0.816	46 000
Otay Creek	7 calendar years	5 610	0.474	43 250
Sweetwater	Last 7 seasons	10 007	0.813	
Sweetwater	26 seasons	11 450	1.387	
Cottonwood Creek at Barrett	26 seasons	19 000†	1.350	73 675
Cottonwood Creek at Morena	26 seasons	9 200†	1.000	46 000
Otay	26 seasons	6 310†	1.350	43 250

* Net—that is, it does not include quantity lost by evaporation from reservoirs.

† Most probable values for long record—see Fig. 17.

TABLE 10.

Source.	95% dry year.	98% dry year.	Railroad Commission.	O'Shaughnessy.
Tuolumne:	Acre-feet.	Acre-feet.		
1 000 000 acre-ft. storage	1 056 000	943 000		
1 500 000 " " "	1 275 000	1 173 000		
1 800 000 " " "	1 380 000	1 275 000		
	Millions of gallons daily.	Millions of gallons daily.	Millions of gallons daily.	Millions of gallons daily.
Eel	180			
San Leandro	6.65	5.14	6	
Temescal	0.266	0.198	0.250	
Crystal Springs	18.5	17.0		
Calaveras	43.24	41.5		Less than 30
San Luis Rey	12.8	12.2		18
Otay	3.1	2.74	2.5	5.16
Morena	4.6	3.86	3.5	8.5 in 1912 6.5 " 1913
Cottonwood Morena, 46 000 acre-ft. Barrett, 27 675 " " 73 675 acre-ft.	6.4	5.43	5.4	19.36 " 1912 15.00 " 1913
Increase yield by Barrett Dam	1.8	1.57	1.9	5.0 " 1913

Mr. Cory. of supply, including the San Luis Rey River at what is known as the Warner Dam site. Mr. O'Shaughnessy, at the same hearing before the Railroad Commission, estimated the safe yield from this source at 13 000 000 gal. daily.*

Table 10 should be considered in connection with Mr. O'Shaughnessy's criticisms of the writer's presentation of the safe yield from local sources based on investigations by several local engineers and Brig.-Gen. H. M. Chittenden (*Retired*), and A. O. Powell, of Seattle; William Mulholland and J. B. Lippincott, of Los Angeles; George G. Anderson, of Denver; Allen Hazen, of New York City; and Leonard Metcalf, of Boston, all Members, Am. Soc. C. E.; the Army Board, and, respecting the sources available for the East Bay cities, accords with his own conclusions.

It is interesting to note in Table 10 that if the available storage in the Tuolumne water-shed is 1 000 000 acre-ft., as estimated by the United States Geological Survey, the safe net yield (95% dry year standard) is only 1 066 000 acre-ft. Using the maximum storage in the basin, as taken by the writer, 1 500 000 acre-ft., the safe net yield is 1 275 000 acre-ft. This storage Mr. Burton Smith considers entirely too high, without exceeding a reasonable cost per unit capacity. Should it be possible to create 1 800 000 acre-ft. of storage in twenty-six reservoirs, as Mr. O'Shaughnessy proposes, the net safe yield would be 1 380 000 acre-ft. The writer, however, knows of no one except Mr. O'Shaughnessy who has ever suggested the possibility of developing such a tremendous quantity of storage.

Mr. O'Shaughnessy quotes from a report by Mr. John R. Freeman, dated March 21st, 1914, to the City Attorney and the City Engineer of San Francisco on "Reservoirs Ultimately Required for the Hetch Hetchy Water Supply of 400 Million Gallons Daily for San Francisco under the Terms of the Raker Bill." The writer has several times had occasion to review hydrographic estimates of Mr. Freeman, and has always arrived at results essentially similar to his. Accordingly, thinking the excerpt given was probably conditioned on assumptions as to irrigation needs, etc., permission to examine the report was asked. He was advised by the City Engineer that the report was not a public document, and permission to see it was refused. Comment on the report or the quotation from it, therefore, is impossible.

The writer is very much surprised that Mr. O'Shaughnessy should express inability to understand the writer's statement that the City had revised the estimated cost of the first 160 000 000 gal. daily installation from Mr. Freeman's figures of $37 501 400 to $64 000 000. The conference between the representatives of the Turlock and Modesto Irrigation Districts and the City Engineer and City Attorney of San Francisco, held in Washington while the Raker Bill was being

* *Ibid*, p. 675.

framed, resulted in several drafts of the bill. One printed on June 23d, 1913, provided that the irrigation districts, before demanding any stored water, should, whenever the Director of the United States Reclamation Service might require, provide additional storage at an initial cost not to exceed $15 per acre-foot of storage capacity. In the bill as passed all provisions requiring the irrigation districts to do anything positive were eliminated, by the House Committee on Public Lands, for legal reasons. However, in discussing this June 23d, 1913, draft of the Raker Bill, it was asked that such limiting cost per acre-foot of storage be raised from $15 to $20. The irrigation districts' representatives declined, pointing out that, according to the Freeman estimate, the construction of the Hetch Hetchy Dam to its initial proposed height would create storage costing only $12.50 per acre-foot, and when continued up to the final height of 300 ft. (capacity then 330 000 acre-ft.), would reduce the cost to $8.50 per acre-foot. The City Engineer denied this, and stated that his revised estimates of the Hetch Hetchy Dam showed the cost to be in excess of $20 per acre-foot, and, further, that his estimate for the entire Freeman first installation was also much greater, and in fact was $64 000 000. These statements were official utterances of the City Engineer at official conferences with official representatives of the irrigation districts. Recently, on being reminded of this by the writer, Mr. O'Shaughnessy answered by saying that the too low unit prices of Mr. Freeman were just about offset by reductions resulting from no irrigation waters being permitted to be brought to the Bay region. However, on page 123 he says: Mr. Cory.

"Thus far, no radical departure from the Freeman plans is proposed, except that it may be considered advisable to install initially a smaller pipe line, 45 miles long, across the San Joaquin Valley for a supply of 60 000 000 gal. per day, and to develop at once 50 000 hydro-electric horse-power."

The differences in cost between 45 miles of pipe with respective capacities of 60 000 000 and 160 000 000 gal. daily, can hardly equal the increase of the City Engineer's estimate of $64 000 000 over the Freeman estimate of $37 501 400.

The writer, the Army Board, and Mr. O'Shaughnessy are in entire agreement as to the very great desirability for San Francisco Bay Cities to determine soon what distant water supply source or sources will ultimately be wanted, and immediately arrange for the necessary water rights. Special legislation, granting to the Bay region the water rights desired and protecting these rights indefinitely until it may be found desirable to develop the sources, can be obtained almost for the asking from any session of the State Legislature. Even if a Constitutional amendment should be necessary, very little more effort would be required, as the sentiment throughout the State is certainly

Mr. Cory.

almost unanimous in favor of assisting important communities to plan intelligently in broad, far-seeing ways for their water needs especially.

Quotations of the cost of developing electrical energy in the hydro-electric and the steam plants, respectively, of the Pacific Gas and Electric System, as set forth in Application No. 400 before the California Railroad Commission, are made as a "refutation of this theory" as to the value of hydro-electric power in the Hetch Hetchy project as expressed by Mr. Grunsky and the writer. The Pacific Gas and Electric Company's load factor is about 62%; the hydro-electric plants are naturally run all the time possible, and have a much higher load factor, and the steam plants a much lower load factor. Hence the load factor of the combined hydro-electric plants is more than three times that of the combined steam plants. It is almost an axiom that, with any given set of conditions, the relative economy of hydro-electric and steam plants is determined by the load factor. As a matter of fact, the writer's comments in the paper concerning the value of hydro-electric power are believed to be entirely fair, candid, and correct.

In several places Mr. O'Shaughnessy, both definitely and by inference, states that the Raker Bill contains provisions requiring the irrigation districts to develop water storage of their own. As already explained, in the original negotiations between the representatives of San Francisco and the irrigation districts, a plan was worked out whereby the irrigation districts were to provide storage as should be required by the Director of the Reclamation Service. The House Committee on Lands, however, decided that Congress, in framing the bill, had power to impose conditions of performance only upon the grantee. Accordingly, all of such provisions were eliminated, and the legislation, as passed by Congress, and the provisions thereof officially accepted by the City of San Francisco, contain no reference or implication requiring the irrigation districts to provide any additional storage.

The following quotations should be read in connection with Section 9 of the Raker Bill itself.

"Moreover, as explained by the Army Board to the Committee on Public Lands, should a controversy ever arise between the City and the irrigationists as to their respective rights, domestic consumption being the highest use to which water can be applied, under the laws of the State of California, the rights of the City will be paramount."

"Pending the construction of additional reservoirs by the irrigation districts, the Raker Bill provides that whenever the districts desire water in excess of that to which they are entitled under the terms of the bill, the City, on the demand of the districts, shall sell such quantities of stored water as may be needed for the beneficial use of the

districts—without impairing the supply for domestic requirements—at such a price", etc. Mr. Cory.

"It is provided, however, that the City may require the irrigation districts to purchase a minimum quantity of stored water, and shall not be required to furnish more than a maximum quantity, in any calendar year, the minimum and maximum quantities to be fixed by the Secretary of the Interior, keeping in mind the superior needs of domestic use."

"It is not within the range of probability that if during any year the City's supply of stored water should be brought so low as to make a water famine imminent, she should still be compelled to furnish water to the districts," etc.

The Raker Bill, though specifically stating that it does not interfere in any way with the laws of the State of California, grants the use of certain lands and rights of way in the Yosemite National Park and the Stanislaus National Forest only upon certain conditions which Congress imposed as entirely extraneous and having no relation whatever to California laws. The text of the bill with respect to these is as follows:

"Sec. 9, that this grant is made to the said grantee subject to the observance on the part of the grantee of all the conditions hereinbefore and hereinafter enumerated:

* * * * * * *

"(*b*) That the said grantee shall recognize the prior rights of the Modesto irrigation district and the Turlock irrigation district, as now constituted under the laws of the State of California, or as said districts may be hereafter enlarged to contain in the aggregate not to exceed three hundred thousand acres of land, to receive two thousand three hundred and fifty second-feet of the natural daily flow of the Tuolumne River, measured at the La Grange Dam, whenever the same can be beneficially used by said irrigation districts, and that the grantee shall never interfere with said rights.

"(*c*) That whenever said irrigation districts receive at the La Grange Dam less than two thousand three hundred and fifty second-feet of water, and when it is necessary for the beneficial use to receive more water the said grantee shall release free of charge, out of the natural daily flow of the streams which it has intercepted, so much water as may be necessary for the beneficial use of said irrigation districts not exceeding an amount which, with the waters of the Tuolumne and its tributaries, will cause a flow at La Grange Dam of two thousand three hundred and fifty second-feet; and shall also recognize the rights of the said irrigation districts to the extent of four thousand second-feet of water out of the natural daily flow of the Tuolumne river for combined direct use and collection into storage reservoirs as may be provided by said irrigation districts, during the period of sixty days immediately following and including April fifteenth of each year, and shall during such period release free of charge such quantity of water as may be necessary to secure to the said irrigation districts such four thousand second-feet flow or portion

Mr. Cory.

thereof as the said irrigation districts are capable of beneficially directly using and storing below Jawbone creek: Provided, however, That at such times as the aggregate daily natural flow of the watershed of the Tuolumne and its tributaries measured at the La Grange Dam shall be less than said districts can beneficially use and less than two thousand three hundred and fifty second-feet, then and in that event the said grantee shall release, free of charge, the entire natural daily flow of the streams which it has under this grant intercepted.

"(*d*) That the said grantee whenever the said irrigation districts desire water in excess of that to which they are entitled under the foregoing, shall on the written demand of the said irrigation districts sell to the said irrigation districts from the reservoir or reservoirs of the said grantee such amounts of stored water as may be needed for the beneficial use of the said irrigation districts at such a price as will return to the grantee the actual total costs of providing such stored water, such costs to be computed in accordance with the currently accepted practice of public cost accounting as may be determined by the Secretary of the Interior, including, however, a fair proportion of the cost to said grantee of the conduit, lands, dams, and water-supply system included in the Hetch Hetchy and Lake Eleanor sites; upon the express condition, however, that the said grantee may require the said irrigation districts to purchase and pay for a minimum quantity of such stored water, and that the said grantee shall be entitled to receive compensation for a minimum quantity of stored water and shall not be required to sell and deliver to the said irrigation districts more than a maximum quantity of such stored water to be released during any calendar year: Provided, however, That if the said irrigation districts shall develop sufficient water to meet their own needs for beneficial use and shall so notify in writing the Secretary of the Interior, the said grantee shall not be required to sell or deliver to said irrigation districts the maximum or minimum amount of stored waters hereinbefore provided for, and shall release the said districts from the obligation to pay for such stored water: And provided further, That said grantee shall without cost to said irrigation districts return to the Tuolumne River above the La Grange Dam for the use of the said irrigation districts all surplus or waste water resulting from the development of hydro-electric energy generated by the said grantee.

"(*e*) That such minimum and maximum amounts of such stored water to be so released during any calendar year as hereinbefore provided and the price to be paid therefor by the said irrigation districts are to be determined and fixed by the Secretary of the Interior in accordance with the provisions of the preceding paragraph.

"(*f*) That the Secretary of the Interior shall revise the maximum and minimum amounts of stored water to be supplied to said irrigation districts by said grantee as hereinbefore provided, whenever the said irrigation districts have properly developed the facilities of the Davis Reservoir of the Turlock Irrigation District and the Warner-Dallas Reservoir of the Modesto Irrigation District to the fullest practicable extent up to a development not exceeding in cost $15 per acre-foot storage capacity, and whenever additional storage has been provided by the said irrigation districts which is necessary to the economical utilization of the waters of said watershed, and also after water losses

and wastes have been reduced to such reasonable minimum as will assure the economical and beneficial use of such water. Mr. Cory.

"(*g*) That the said grantee shall not be required to furnish more than the said minimum quantity of stored water hereinbefore provided for until the said irrigation districts shall have first drawn upon their own stored water to the fullest practicable extent.

"(*h*) That the said grantee shall not divert beyond the limits of the San Joaquin Valley any more of the waters from the Tuolumne watershed than, together with the waters which it now has or may hereafter acquire, shall be necessary for its beneficial use for domestic and other municipal purposes.

* * * * * * *

"(*j*) That by 'the flow,' 'natural daily flow,' 'aggregate daily natural flow,' and 'what is naturally flowing,' as are used herein, is meant such flow as on any given day would flow in the Tuolumne River or its tributaries if said grantee had no storage or diversion works on the said Tuolumne watershed."

It will be noted that Paragraph (*f*) does not require the construction of any storage by the Turlock and Modesto Irrigation Districts, but, instead, requires the Secretary of the Interior to revise the maximum and minimum quantity of stored water referred to in Paragraph (*e*) whenever, and if, certain things occur.

One further excerpt is given from Mr. O'Shaughnessy's discussion:

"The Hetch Hetchy scheme has been the subject of very serious consideration by many engineers who, after comparing it with all other available sources of water supply, have endorsed it. The engineers who have questioned it, with the sole exception of those employed by the irrigation districts, have been the representatives of special competing interests, hoping to dispose of their own private holdings for their own private gains."

Mr. Van Norden expresses a radically different opinion. It certainly would be interesting, and doubtless very helpful to the local communities, to know what the consensus of opinion among local engineers really is. The writer suggests that Messrs. O'Shaughnessy and Van Norden co-operate in arranging for the San Francisco Association of Members of the American Society of Civil Engineers to carry out a confidential poll of the local Members and Associate Members of the Society—of which there are more than 130—or better yet, of all the societies which joined in holding the recent International Engineering Congress.

Turning now to the criticism, by Messrs. Rhodin, Sloan, and Brown, of the presentation in the paper of the Eel River source, it may be said that Eel River was given about as much attention as the Army Board gave it. Incidentally, the safe yield of this source, as given in Table 10, is exactly that found by the Army Board, which, however, also finds that with a conduit of 225 000 000 gal. daily capacity, and utilizing considerable local storage, the safe yield could be

Mr. Cory. increased to 200 000 000 gal. daily. The writer has not checked these figures. Should the East Bay cities continue to keep aloof from San Francisco and its water plans, as Mr. Sloan suggests, they would probably find Eel River the most attractive and available mountain supply—possibly at the lowest cost per thousand gallons of any distant water under any arrangement.

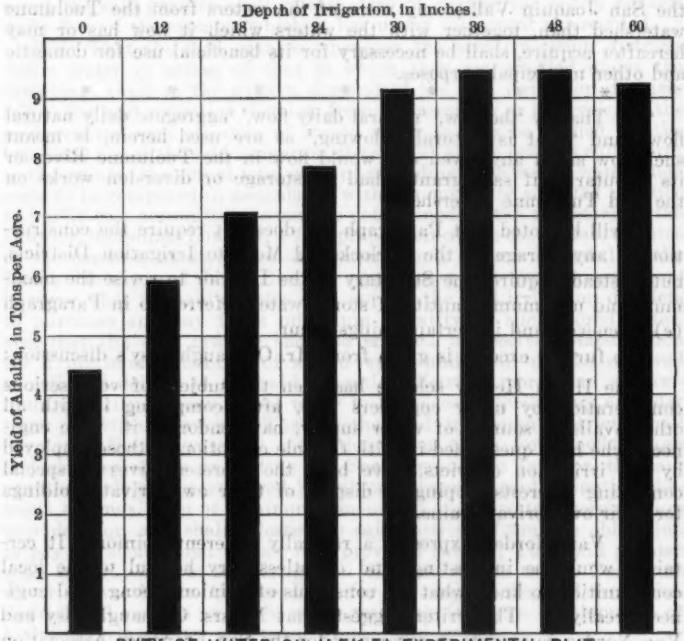

FIG. 29.

Mr. Grunsky and others refer to the local sentiment regarding Sierra water. It undoubtedly is strong, which is interesting, and so is the reason for its existence. Sacramento, situated at the very edge of the Sierras' western foot-hills, is overwhelmingly in favor of filtered Sacramento River water. Human nature in the two cities is doubtless practically the same, so that the difference in the public sentiment in this regard is doubtless due to the respective com-

munities having had engineering advice along different lines. The real enthusiasm with which San Francisco people greeted Mr. Grunsky's Tuolumne project in 1902 was probably because it was the first concrete plan put forward under the new charter, whereby the "yoke" of the Spring Valley Water Company might be thrown off and that corporation crumpled. Mr. Cory.

The writer has been much interested in the expressions of opinion concerning the water needs of the 300 000 acres in the Turlock and Modesto Irrigation Districts. However, summarizing the essential facts, concerning which there seems to be general consent, it appears that the adaptation of most of the area is alfalfa, dairying, and stock; that six irrigations per annum are required for maximum economical yield; that the soil is, broadly speaking, rather sandy; and that the mean annual rainfall over the territory is 11 in., of which little, and frequently none, occurs in the cropping season.

Consequently, the writer sees no occasion to modify his ideas on the subject expressed in the paper, and especially the opinion that ultimately the total irrigation needs per gross acre in this area and in the Imperial Valley will prove to be practically the same—about 3.33 acre-ft. In reference to this matter, Fig. 29* is instructive.

Since writing the paper, more than a year and a half ago, the following important developments in the water situation have occurred:

1.—The jurisdiction of the Railroad Commission has been extended to the control of all public utilities, inside as well as outside of municipalities.

2.—The city officials and the Spring Valley Water Company agreed upon a sales price for all the latter's property which it was deemed advisable to purchase. At a special election, the proposition received a majority but not the necessary two-thirds affirmative vote.

3.—An election was held in the East Bay cities for the formation of a municipal water district, but the proposition was defeated.

4.—The Richmond Municipal Water District voted against authorizing a bond issue for bringing water from the Sacramento River.

5.—The Marin Municipal Water District has condemned before the Railroad Commission the property of the Marin Water and Power Company and the North Shore Water Company. The companies, feeling that the price fixed by the Railroad Commission was too low, have appealed to the State Supreme Court.

* This is Plate 4 of *Bulletin No. 1*, "Progress Report of Co-operative Investigations in California, 1912–1914" by Frank Adams, Irrigation Investigations, Office of Experiment Stations, United States Department of Agriculture, published by the California State Department of Engineering, 1915.

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Paper No. 1353

THE PICAZA BRIDGE

By A. A. Agramonte, Assoc. M. Am. Soc. C. E.

Synopsis.

This paper contains a description of a reinforced concrete bridge spanning a part of the Valley of the Salada River, in the Province of Buenos Aires, Argentine Republic, and, together with ten other bridges and some smaller works, was built under the direction of the Drainage Commission of the Province of Buenos Aires.

The bridge consists of a central span of 26 m. (85 ft.) and six lateral spans of 28.5 m. (93.5 ft.) each. The central span consists of a floor system suspended by steel tension members from two concrete arches each of which is reinforced with a structural steel skeleton arch, and tied by a steel chord to take up the thrust. The hangers are bolted to the chord, and all the tension members are encased in concrete. Roller bearings at one end provide for expansion and contraction. Two transverse arches supply wind bracing and help to make the structure rigid.

Each lateral span consists of three continuous reinforced concrete beams resting on three supports. The central supports are concrete pylons, and are fixed. The end supports are masonry pillars; and steel bearing plates, on pillars and beams, face to face, allow free temperature movement. The floor and sidewalk are carried on a transverse beam and slab system.

A brief description is given of the drainage works and the structures involved, and then the Picaza Bridge is described, the working organization, the foundations, the materials, the labor, the finish, the progress, and the costs.

General Description of the Drainage Works.

The Region.—The Drainage Commission of the Province of Buenos Aires is a board of land owners having charge of the design and execution of works for the relief of the southern part of that Province, a region subject to inundation at intervals of from 10 to 15 years. This region is a great plain dipping gradually toward the Atlantic Ocean on the east, and receiving the discharge of the flashy hill streams, or "arroyos", which drain the uplands and "sierras" on the west. Below this upland region watercourses disappear from lack of fall, and, in the whole area subject to inundation, there is only one river, the "Rio Salada", with a continuous channel, and this is utterly inadequate to convey the flood flow from its own water-shed. The area subject to inundation is approximately 39 750 sq. km., or more than 9 700 000 acres.

The Project.—To ameliorate the disastrous effects of the periodical inundations in a country with no natural drainage channels, a number of canals have been projected and built for the double purpose of, first, carrying the average flood flows of the "arroyos" directly to the sea at a relatively high level, and secondly, receiving the drainage of the lower zones through numerous sluice-gates at a low level after the discharge of the flood flow of the "arroyos". Complete flood prevention would be economically impossible. The system takes a normal flow of 46 000 000 cu. m. per day (18 800 cu. ft. per sec.).

These canals are of many sections and lengths, but a description of them would be foreign to the object of this paper. It is sufficient to state that they consist in general of a central excavated section from 2 to 4 m. (7½ to 15 ft.) deep and from 10 to 30 m. (33 to 108 ft.) wide and a flood section formed by lateral dikes from 1.5 to 4 m. (5 to 13 ft.) above the ground level, and from 80 to 300 m. (262 to

984 ft.) in effective width between the dikes. Three of the canals are more than 130 km. (80 miles) long, one is more than 260 km. (160 miles) long, and there are sixteen others of varying lengths, and thousands of miles of laterals. The total length of the main canals is 1 005 km. (624 miles) and the contents are more than 30 000 000 cu. m. (39 240 000 cu. yd.). The total cost to date has been $40 000 000 of official paper money, equivalent to more than $17 400 000 in gold.

Control and Accessory Works.—To discharge flood waters under control, and conserve the natural watercourses where convenient, structures of the following types were built:

1.—The works at the entrance of the derivation canals consisted of entrance gates controlled by machinery, and waste-weirs or gates (generally automatic) to discharge the excess flow to natural channels. The gates and appurtenances are of steel and bronze; the structures are of reinforced concrete with steel skeletons, hoop-bound brickwork or hardwood ("quebracho"), according to location. Piling and sheet-piling were used where necessary. Works of this type have been constructed at the origin of nine canals.

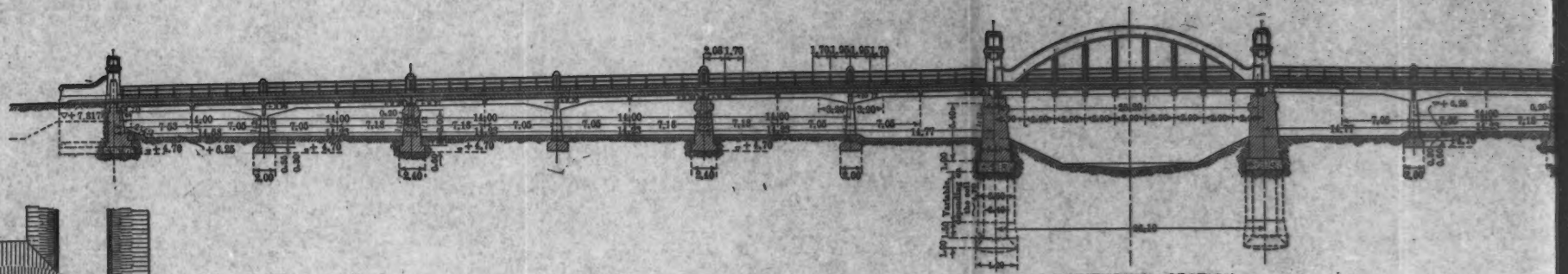
2.—Sluice or drainage gates were placed at appropriate places in the length of the canals, and provide for the drainage of the low-level zone. The gates and appurtenances are of steel and bronze, but hardwood gates are also used. All types are automatic. There are 114 of these structures.

3.—Relief openings are provided with gates operated by machinery, and serve to supply or to empty large "lagunas" or natural reservoirs. The gates, etc., are of metal and the structures are mainly of reinforced concrete. There are five of these openings.

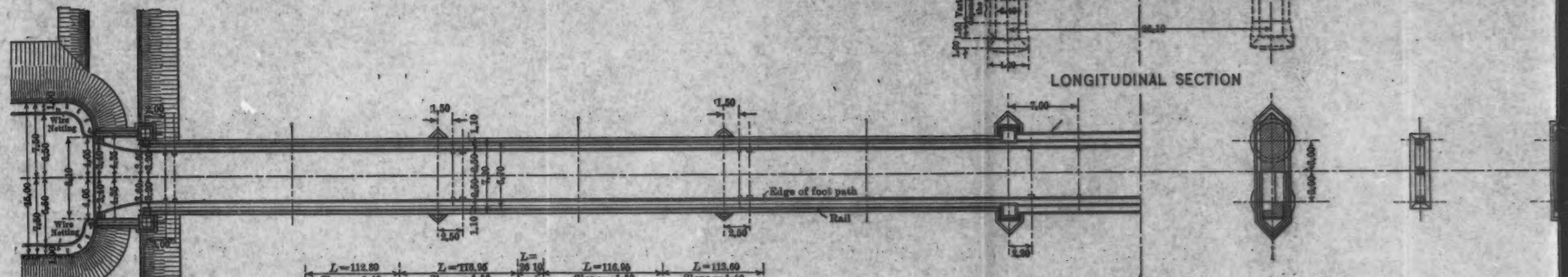
4.—Waste-weirs take the water from the canals to natural watercourses, at places of strategic importance. These are automatic, and generally of reinforced concrete or hoop-bound brickwork. There are six of these weirs, exclusive of those at the head-works.

5.—Inverted siphons are used to pass low-level streams under the canal where it has been found necessary to maintain such channels for the drainage of certain districts. There are three of these.

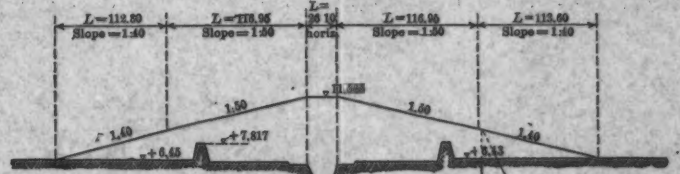
6.—Bridges have been built wherever traffic has required them. They are of varying spans, types, and materials. Table 1 gives information regarding all the bridges built.



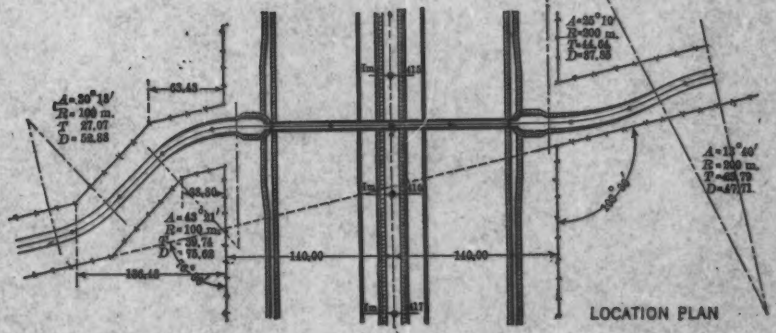
LONGITUDINAL SECTION



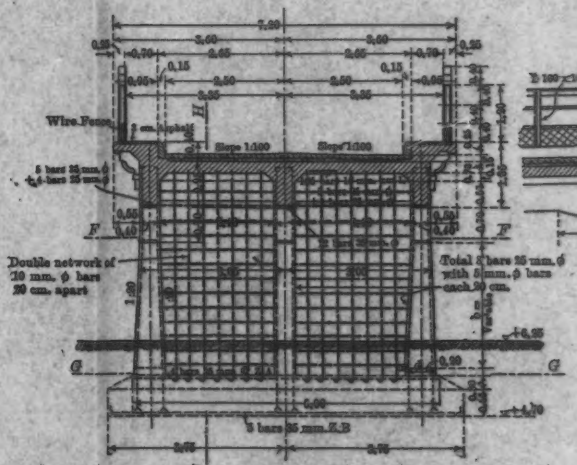
PLAN



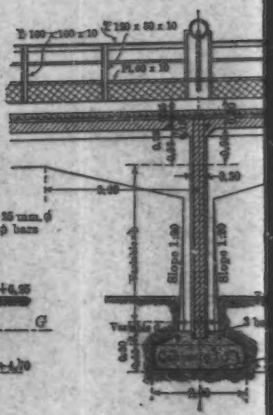
LONGITUDINAL SECTION



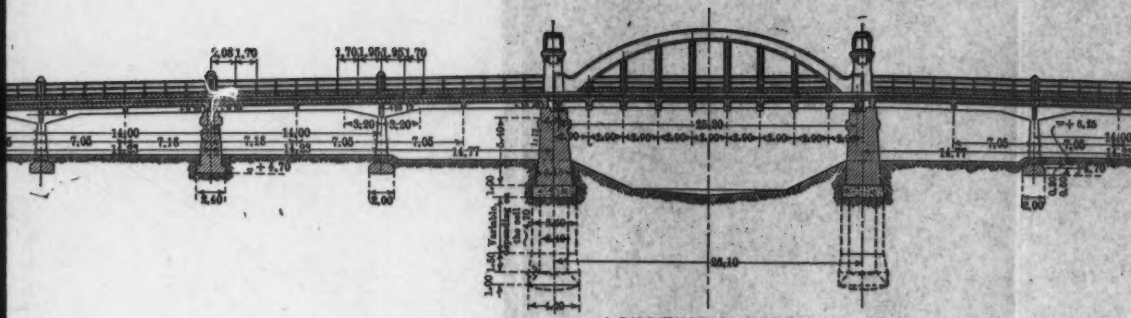
LOCATION PLAN



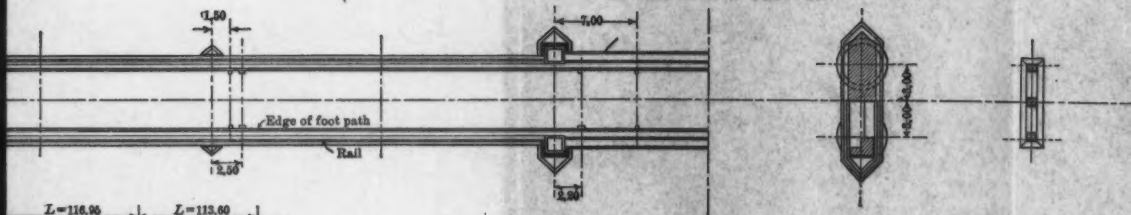
SECTION OF PYLON



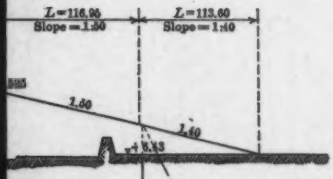
SECTION A-A



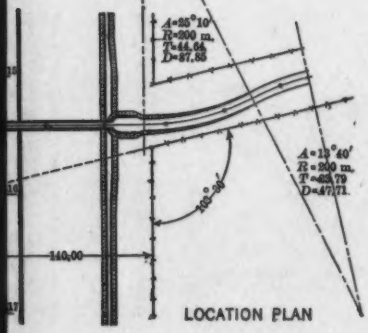
LONGITUDINAL SECTION



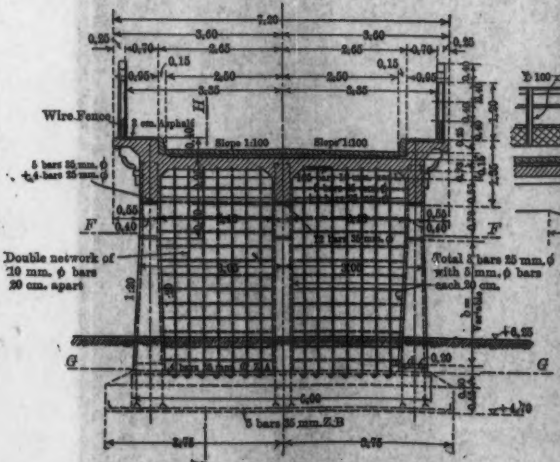
PLAN



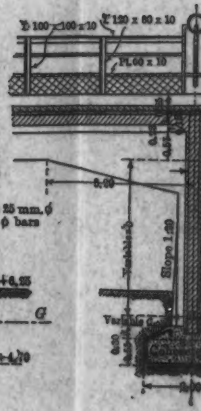
L SECTION



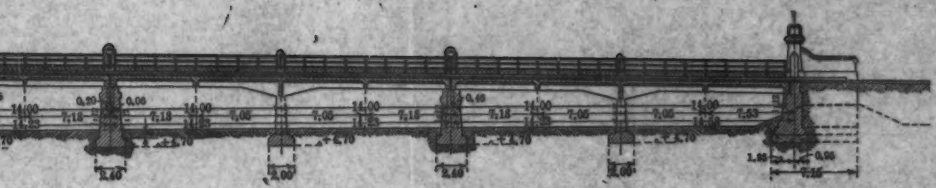
LOCATION PLAN

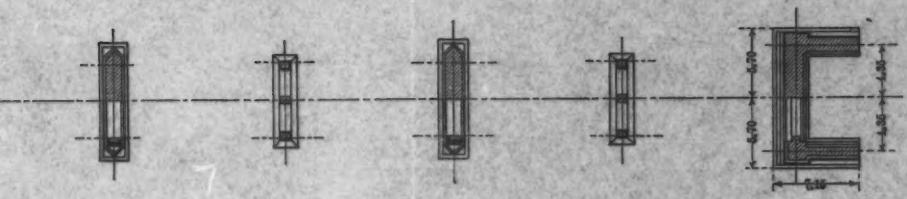


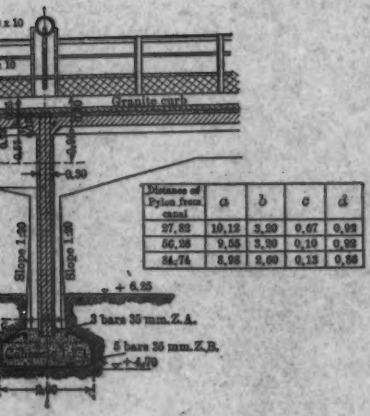
SECTION OF PYLON



SECTION







Distance of Pylon from canal	a	b	c	d
27.82	10.12	3.20	0.67	0.92
56.28	9.55	3.20	0.10	0.92
84.74	8.98	2.60	0.13	0.86

SECTION F-H

GENERAL PLAN
PICAZA BRIDGE
OF THE
DRAINAGE COMMISSION
OF THE
PROVINCE OF BEUNOS AIRES
ARGENTINA

The Picaza Bridge.

Location.—The Picaza Bridge, 5 km. (3 miles) from the City of Dolores on the highway to Sevigné, in common with all the works described, was designed by the Technical Office under the supervision of the late Engineer Director, Mr. Charles Nyströmer. The execution of the work, together with that of ten other bridges and minor structures, was assigned to the writer under Mr. Humphrey W. Cooper, Chief Engineer. The Inspector General of all the works of the Drainage Commission was Mr. Leonard Cooper.

General Description.—The bridge is composed of one central span of 26 m. (85 ft.) and six lateral spans of 28.5 m. (93.5 ft.) each, all in reinforced concrete. The central span consists of a floor system suspended by steel tension members from two concrete arches each of which is reinforced with a structural steel skeleton arch and tied by a steel chord to take up the thrust. The hangers are bolted to the chord, and all the tension members are encased in concrete. Roller bearings at one end enable the arch to move with variations of temperature. Two transverse arches provide for wind bracing and help to give complete rigidity to the structure.

Each lateral span consists of three longitudinal continuous reinforced concrete beams resting on three supports; the central support is a concrete pylon, and is fixed. The end supports are masonry pillars. Steel bearing plates on pillars and beams, face to face, allow free temperature movements. A transverse beam and slab system carries the floor and sidewalk.

Organization.—As far as possible, the work was done by using both piecework and bonus systems. Ordinary day labor was used where unavoidable on a very small proportion of the whole. The writer, although having under construction many other bridges at the same time, made a point of directing all important work personally, a difficulty which may be realized when it is stated that it required a two days' trip to reach some of the other bridges. A general foreman and a time and storekeeper completed the permanent force in charge of the work.

Those who examine the statement of costs of this bridge should bear in mind that the general foreman had to be instructed in his duties while on the work, as no sober, experienced, and reliable con-

TABLE 1.—Bridges Built by the Drainage

Canal No.	Distance from mouth, in kilometers.	Distance to cart materials, in kilometers.	Foundations. Abutments.	Pillars.
1	30	40	Hardwood piles	Steel cylinders
1	65	15	" "	Concrete cylinders
1	106	40	Slab concrete	Slab concrete
2	4	4	Hardwood piles	Steel cylinders
2	36	36	Slab concrete	Slab concrete
2	62	15	" "	" "
2	88	15	" "	" "
2	107	20	" "	" "
3	11	20	Steel piles	
5	6	20	Cylinders, reinforced concrete	
5	34	35	Slab, reinforced concrete	Cylinder. Reinforced concrete
9	4.6	35	Hardwood piles	
9	41	5	Slab, reinforced concrete	Cylinder. Reinforced concrete
9	81	40	Slab concrete	Slab concrete
9	102	30	" "	" "
9	141	25	" "	" "
10	....	35		
15	11	30	Cylinders	Screw piles
15	33	15	Caissons	
18	6	45	Hardwood piles	Hardwood piles
18	17	35	Reinforced concrete cylinders	
2†	4	4	Hardwood piles	
2†	37	37	" "	

Note: Ten more bridges are to be built. * Brick plastered

crete men having the requisite knowledge and skill were to be found in Buenos Aires when the work was commenced. Further, laborers in summer desert the work to go to harvesting, with the expectation of higher wages. Form carpenters, also, have to be taught on the work, and, as they know that new men do not acquire the necessary experience in a short time, they soon become very urgent in their demands for increased wages. In brief, reinforced concrete construction was a continual struggle, first to teach the men and afterward to hold them on the work, and this was aggravated by the difficulty of obtaining reliability in foremen and timekeepers, a greater problem than it may seem from a distant viewpoint.

For the foregoing reasons labor costs should be higher than in regions more favored in the respects indicated; nevertheless, the writer may claim that they represent fair costs for that part of the country, as the work was done for 30% less than the lowest prices bid for it by a contracting firm, and 50% less than the next highest bid (including all administrative charges, etc.). The costs given

COMMISSION OF THE PROVINCE OF BUENOS AIRES.

ELEVATION.		Superstructure.	Span (over all), in meters.
Abutments.	Pillars.		
Stone.....	Steel and concrete.....	Steel cantilever. 3 openings	114.80
"	Stone.....	" " "	90.00
"	"	" " "	73.50
Brick, plastered *.....	Brick, plastered *.....	" " "	73.50
" "	" "	" " "	145.08
" "	" "	" " "	145.08
" "	" "	Steel truss.....	97.40
" "	" "	" "	97.40
Reinforced concrete..	.....	Steel trestle.....	37.10
Brick, plastered *....	.....	Steel truss, one span.....	44.00
" "	Brick, plastered *.....	Steel cantilever. 3 openings..	145.00
" "	" "	One steel truss, two of reinforced concrete.....	72.40
" "	Brick, plastered, and reinforced concrete.....	Reinforced concrete viaduct..	197.50
" "	Brick, plastered *.....	Steel cantilever. 3 openings..	143.90
" "	" "	" " "	143.90
Stone.....	Stone.....	" " "	114.80
Steel.....	.....	Steel trestle.....	.....
Brick, plastered *....	Brick, plastered, and steel	Steel trestle and truss.....	145.50
" "	Brick, plastered, and reinforced concrete.....	Steel cantilever. 3 openings..	145.10
Hardwood.....	Hardwood.....	Hardwood.....	15.00
Reinforced concrete..	.....	Reinforced concrete. 1 span.	13.00
Brick, plastered *....	.....	3 spans. Reinforced concrete	18.20
" "	.....	" " "	14.20

to imitate stone. † On branch canals.

may be taken as representative of work of this class in the Province of Buenos Aires at the present time.

Excavation of Foundations.—The excavation was in hard clay, which was shoveled into Decauville cars and transported by horse traction about 200 m. (650 ft.) to the lateral dikes of the canal. The spoil from the cylinder excavation followed the same route.

Foundations.—The foundations of the lateral piers and abutments were of 1:3:5 concrete, with crusher screenings instead of sand. The materials were brought from stock piles, about 100 m. (328 ft.) from the axis of the bridge, in Decauville cars to a portable mixing platform placed as near the work as possible. The stone and screenings were measured in the cars. The concrete was mixed by hand and placed directly from the platform. Two mixing platforms were used, and were moved as needed. The stone had to be washed well before it was used, as it had been stored in an exposed position for more than a year, and was full of dirt. Water was provided by a windmill pumping to two tanks having a capacity of 12 cu. m.

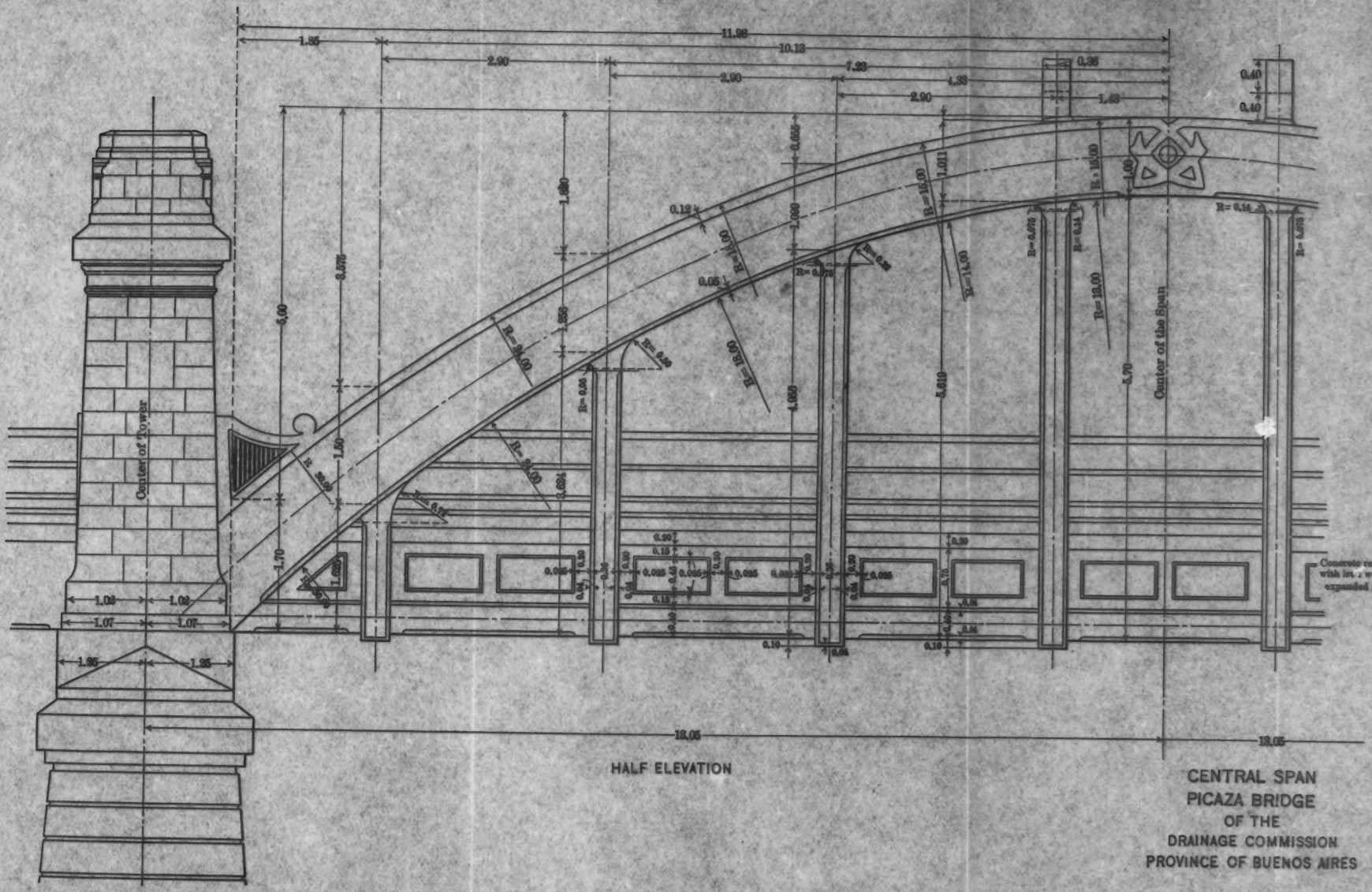
(3 170 gal.), and then conveyed in a pipe line, parallel to the bridge, with outlets at intervals of 14 m. (46 ft.).

The foundations of the pylons are of reinforced concrete, and in these the materials and their proportions are the same as in the plain concrete. In all the foundation concrete, granite rubble was embedded in order to economize in crusher stone, because, owing to a variation in the original design, a large surplus of this material was available.

Each pier supporting the central span rests on a reinforced concrete slab, and this rests on two reinforced concrete cylinders which were sunk to hard clay 9 m. (29½ ft.) below ground level. The internal diameter of each cylinder, at the steel cutting edge, is 3 m. (9 ft. 10 in.), decreasing to 2 m. (6 ft. 6¾ in.) at the top. The lower section of each cylinder was built to a height of 3 m. (9 ft. 10 in.) and at once sunk to the ground-water level, then the upper section was built and the sinking completed. The several steps in the work were as follows:

1.—Excavation, to ground-water level, of area of slab;
2.—Construction of lower section of Cylinder I;
3.—Construction of lower section of Cylinder II; (the wooden framework over the excavation for handling buckets, etc., was erected while the cylinders were being built);
4.—Sinking Cylinder I and then Cylinder II to sand (2 m. below ground-water level), using picks, and shoveling the earth to the platform and to cars, a hand-diaphragm pump being used for unwatering;
5.—Construction of upper sections of Cylinders I and II;
6.—Sinking Cylinder I below the 2-m. level, using two buckets suspended from a pulley raised alternately by a hand-winch, the buckets dumping directly into cars, the unwatering being done with a 6-in. centrifugal pump with a steam motor.

Two men below kept the buckets in continual operation, and a crew of three alternated at the winch and dumped the buckets into the car. In ordinary hard but sandy soil the progress was about 1 m. of depth per day. There were no difficulties until, at a depth of 4 m. (13 ft.) below the ground-water level, quicksand was encountered. At the end of each pumping period, after a little excavation, there



HALF ELEVATION

CENTRAL SPAN
PICAZA BRIDGE
OF THE
DRAINAGE COMMISSION
PROVINCE OF BUENOS AIRES

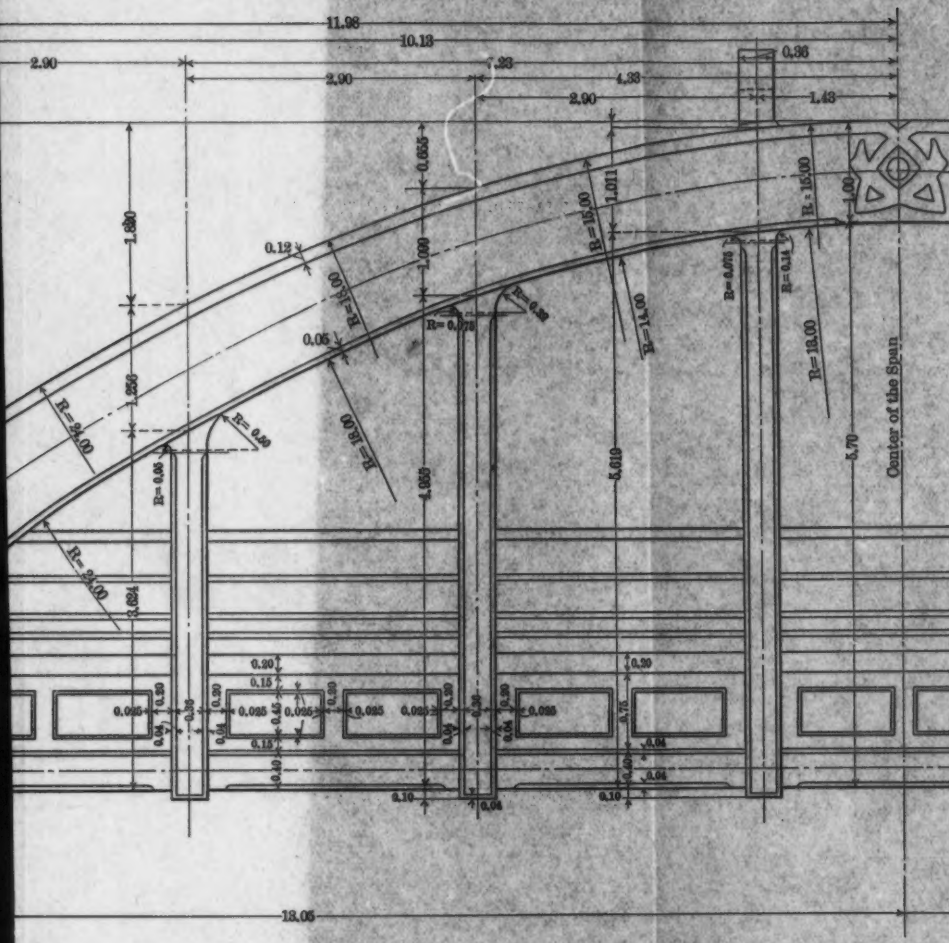
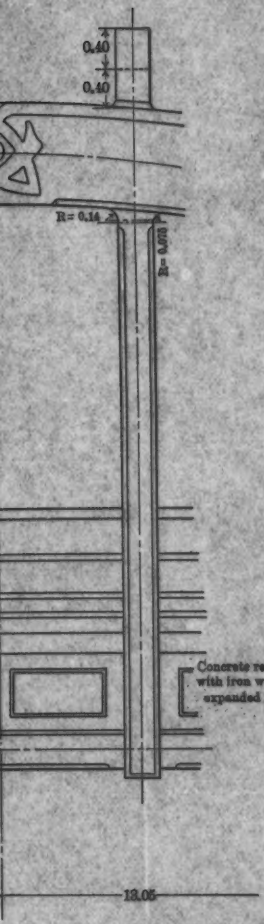
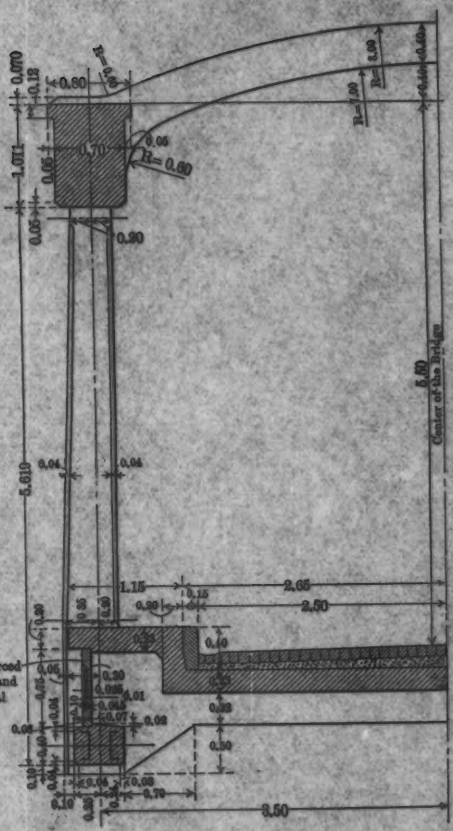

11.98
10.13
2.90
2.90
.23
2.90
4.33
0.36
1.43
1.680
1.850
1.824
0.655
1.000
1.011
0.12
0.05
R= 15.00
R= 15.00
R= 14.00
R= 0.78
R= 0.50
R= 0.50
R= 0.50

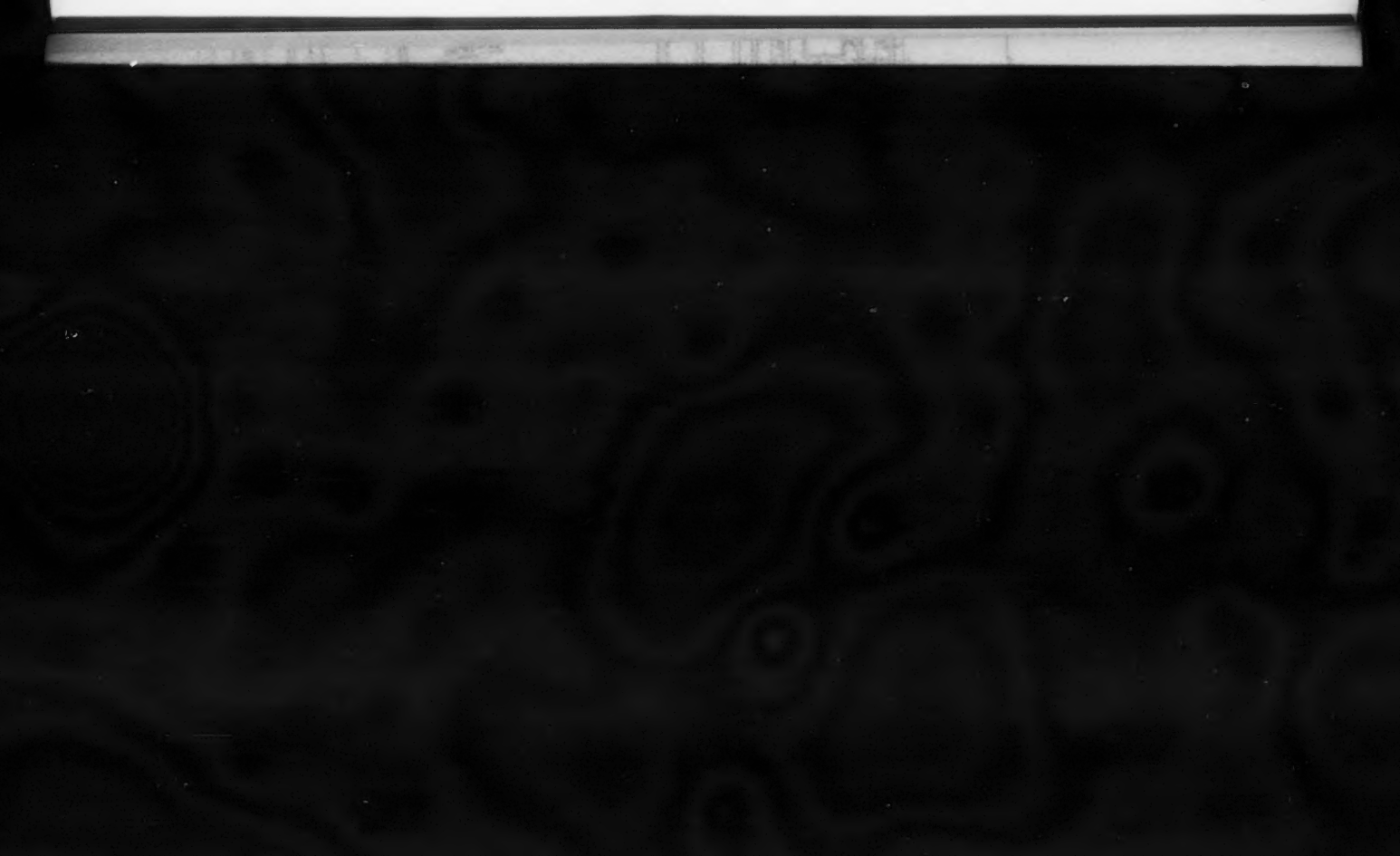
PLATE III.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXX, No. 1353.
AGRAMONTE ON
THE PICAZA BRIDGE.



CENTRAL SPAN
PICAZA BRIDGE
OF THE
DRAINAGE COMMISSION
ROVINCE OF BUENOS AIRES



HALF TRANSVERSE SECTION



was an inrush of fine sand which flowed like mud and filled the excavation at least 1 m. in depth. This material choked the flexible suction hose of the pump and caused delay. The constant inflow of sand began to undermine the upper crust and great fissures radiating from the cylinder excavation began to appear. Progress was stopped, although only 1 m. of fine sand separated the cutting edge from the hard clay; and, though the writer himself entered the water, it was difficult to restore the "morale" of the men.

The remedy for the trouble was simple. An inner cylinder of galvanized-iron sheeting was driven around wooden bracing rings, each sheet being driven into the hard clay. The inner cylinder was well cleaned, the pump suction pipe was hoisted, and the lower 2 m. (6½ ft.) of the cylinder concreted under water, using a wooden tremie to place the concrete, the tremie being raised by the hand-winch when necessary. The upper section was packed with sand and capped with concrete.

A reinforced concrete platform resting on the two cylinders completed the foundation.

Abutments and Piers.—The abutments and piers are of brick masonry, with 1:3 mortar. In part of the work washed stone crusher dust was used instead of sand, and with satisfactory results. In the central piers about 20% of the volume was rubble masonry filler, owing to the surplus of rubble already mentioned.

Pylons.—The pylons are of 1:2:4 reinforced concrete, using 1 to 2-in. stone. They were built simultaneously with the piers, and all were at least 1 month old before the forms for the superstructure were erected. The columns were built in one continuous operation, allowing time for thorough settling of the wet concrete by working on two or more alternately, pouring sections 1 m. (3¼ ft.) high at a time.

Superstructure.

Forms for Lateral Spans.—The forms were designed to obtain the greatest recover value for use elsewhere. Two complete sets (each for one 28-m. (91.86-ft.) span) were used in rotation with three sets of uprights. This arrangement allowed for continuous work by the carpenters.

Movement of Materials.—Sand and stone were measured in the Decauville cars at the stock piles and transported to the central part

of the bridge where two low-level mixing platforms received the stone after it had passed over an inclined slatted platform on which it was washed thoroughly with a hose. The sand was discharged directly to the platforms. Arrangements were made to permit the discharge of sand or stone to either platform, and the stone-washing crib or inclined platform was divided into four compartments, each having a capacity of 1 cu. m. (35.3 cu. ft.) so that washing could be carried on always two batches ahead of the mixing. No difficulties were experienced at any time with this arrangement.

Cement and sand, in the proportions of 1:2, were mixed by hand, dry, turning the mixture three times, until it was of uniform color; then two parts of washed stone were added, and the concrete was mixed with plenty of water until the mass was of uniform jelly-like consistency.

A 50-ft. steam derrick with a bucket containing ½ cu. m. (11.7 cu. ft.) received the concrete directly from the mixing platform and hoisted it to an elevated distribution trestle parallel to the bridge.

The concrete was dumped from the bucket of the derrick into a Decauville car on the trestle, and the car ran by gravity to the point of discharge, where it was placed by a movable dumping box, 2 by 2 by 0.30 m. (6.56 by 6.56 by 1 ft.), with adjustable, steel chute (a 0.30-m. smokestack sawed in two). There was no trouble of any kind with this arrangement, and concrete was in place within half an hour of mixing. Two men handled the car and derrick bucket, and dumped the concrete into the dumping box; two men below tamped the concrete, and controlled the discharge of the chute; and a fifth man acted as general emergency man, a very important position, as the conditions of discharge varied continually. It may be noted that all concrete was mixed very wet, and all forms were drenched before they were filled, although, of course, all surplus water was drained off.

One elevated trestle was used for one-half of the bridge, and after concreting, it was taken down and re-erected for the other half. The truck was about 2 m. (6.56 ft.) above the bridge floor and about 1 m. (3.28 ft.) from the bridge railing over which it was necessary to chute the concrete. The concrete was in the proportions, 1:2:2. The sand was a very good, coarse, river sand from Uruguay. Fine granite chippings were used for the lower parts of the beams, where

FIG. 1.—PYLON COLUMNS AND BRICKWORK PIERS AND ABUTMENTS, PICAZA BRIDGE.—SECOND MONTH.

FIG. 2.—FORMS AND REINFORCEMENT OF HALF OF ONE LATERAL SPAN BEFORE ADJUSTING REINFORCEMENT.

the complicated reinforcements demanded a very fine aggregate, and a 1 to 2-in. stone in the remainder of the concrete.

With one gang of 10 men working alternately on the two mixing platforms, it took 3 days to concrete one 28-m. (91.86-ft.) span, the work being executed in the following order:

1a.—All reinforcement was securely placed, tied, and braced.

b.—All forms were washed and well drenched.

c.—All concrete adhesion surfaces (pylons) were picked and grouted with neat cement.

d.—All surfaces of contact over masonry pillars were rubbed with clay to prevent adhesion.

e.—Sliding bearing plates for beams to rest on pillars were placed and greased face to face.

2.—First Day.—Longitudinal beams were concreted from the pylon outward, on both sides to the center of the 14-m. (45.93-ft.) span, leaving a vertical joint near the center of each beam.

3.—Second Day.—The vertical joints were washed, brushed lightly, and grouted with neat cement. The beams were finished to the end of the span.

4.—Third Day.—After riveting the railing and adjusting the horizontal hand-rail to exact position (it is embedded in the concrete of the sidewalk), the slab was concreted, beginning at one end and finishing at the other.

Great care was taken throughout to ensure bond by washing and grouting, and every precaution was taken against the displacement of reinforcement or of concrete already set.

Central Span.—Entirely new forms were required for the central span. The steel reinforcing arch was utilized as much as possible to support the forms for the arch, and the transverse beams and slabs were supported from below.

The work was executed in the following order:

1.—The steel reinforcing arch was erected and bolted. This was done with the customary falsework, which was used subsequently to support the floor forms. The arches were braced by wooden struts and tied with wire while being adjusted and bolted.

2.—The forms for the concrete arch and suspended floor system were built, using material from the old forms where possible.

3.—The arches were concreted with 1:2:1½ concrete, using fine stone. One arch was completed in one day, beginning at the haunches and working toward the crown, with alternate pourings, so as to distribute the weight symmetrically, thus no construction joints were necessary.

4.—The floor-beams, slabs, and sidewalks were concreted in the order named, and the pavement and sidewalk finished as that of the rest of the bridge.

5.—One month after concreting the arches, in order to allow for subsequent deflections, and with all dead load in place (paving block, sand, etc.), the hangers or suspension members were concreted, and afterward the longitudinal tension members.

After concreting the lateral span of the first half of the bridge, the masonry on that side was continued upward and, at the same time, granite curbing and compressed asphalt tile sidewalk were placed.

Pavement.—The concrete floor was water-proofed with a 1-cm. (0.39-in.) coating of a coal-tar sand mixture placed hot and ironed. All expansion joints were filled with asphalt. Surface and sub-surface drains were placed, and the floor of the bridge was paved with Swedish granite block paving sets, consisting of 0.10-m. (3.9-in.) blocks, placed so as to form a mosaic fan-shaped design. These blocks were laid on a 7-cm. (2¾-in.) cushion of sand, and the joints were filled with coal-tar after the pavement had been consolidated thoroughly.

The abutments were paved with ordinary 15-cm. (5.9-in.) block paving resting on a concrete raft slab, to allow for subsequent settling. Although the fill on which the pavement rests was consolidated thoroughly by watering and tamping, it had settled 5 cm. (2 in.) one year after it was laid. It had been given a super-elevation of 7 cm. (2¾ in.).

Concluding Masonry, Ornamental Designs, etc.—After concreting the central span, the masonry of the central pillars was finished and the ornamental designs at the crowns of the arches and at the haunches were added by grouting behind plaster of Paris negatives held in place by bracing and wiring. Granite mosaic work was used to fill the decorative triangular panels above the haunches of the arches.

FIG. 3.—CENTRAL SPAN, MELAN REINFORCING ARCH. ELEVATED TRACK TRESTLE FOR END SPANS, PICAZA BRIDGE.

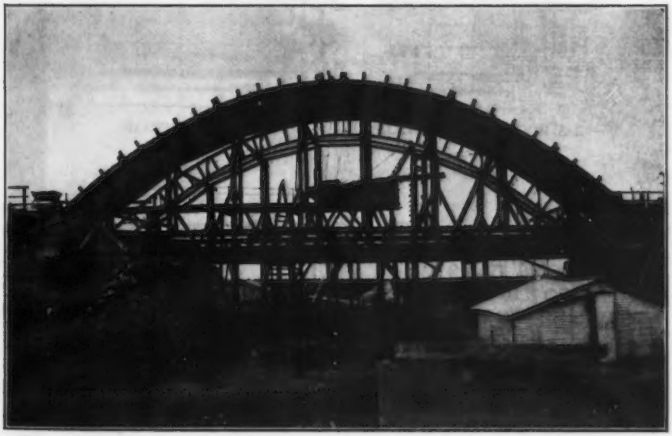


FIG. 4.—CENTRAL SPAN, PICAZA BRIDGE. FORMS FOR ARCH.

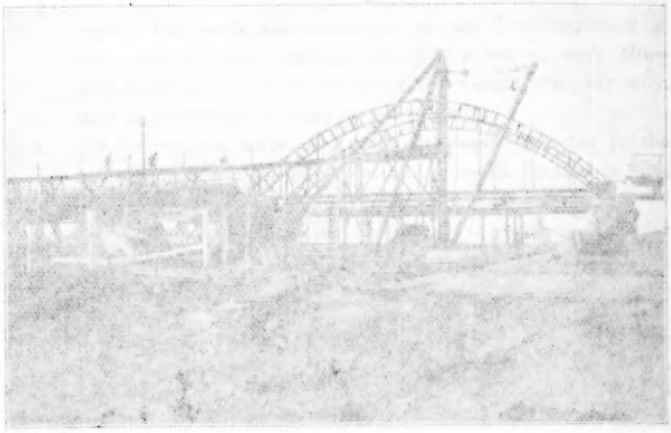
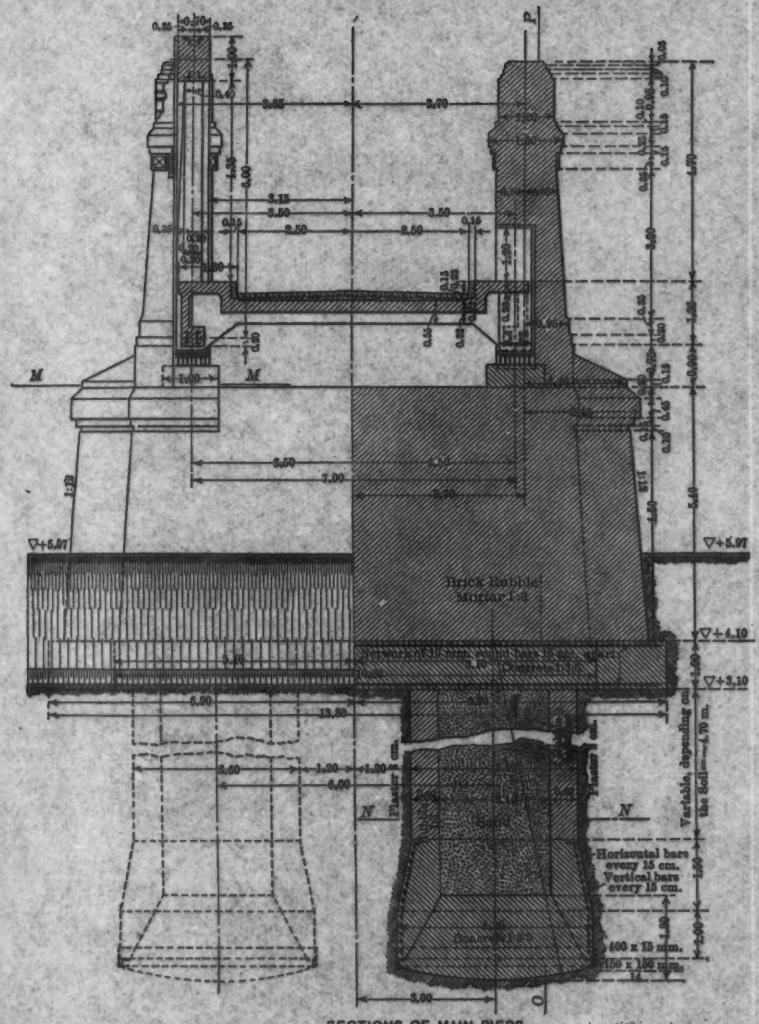
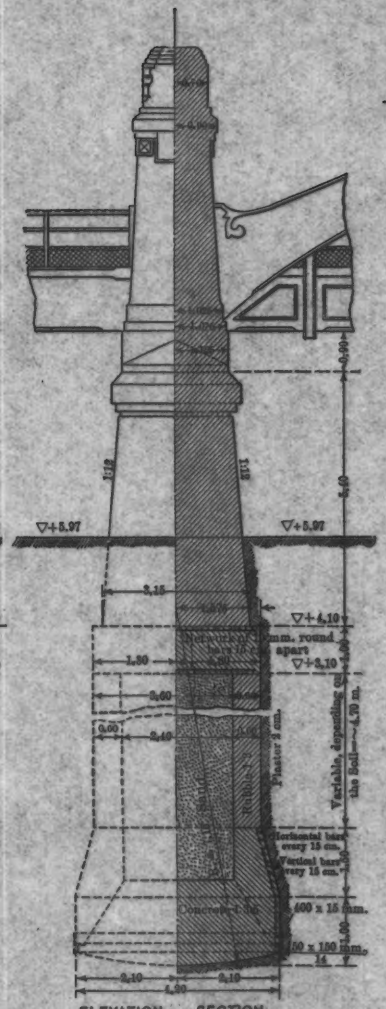


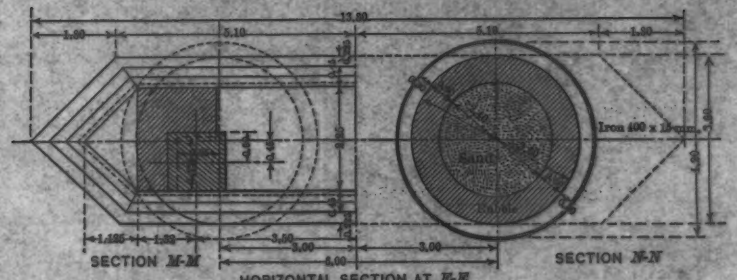
PLATE IV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXX, No. 1355.
AGRAMONTE ON
THE PICAZA BRIDGE.



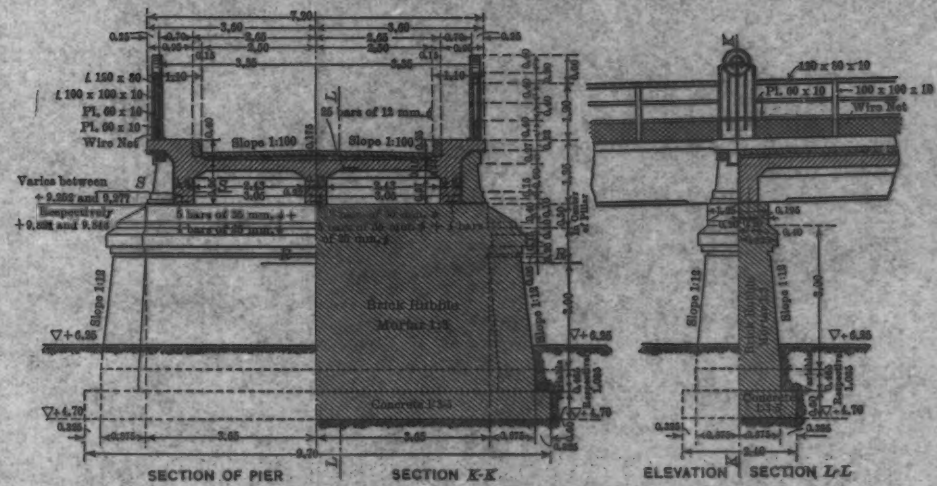
SECTIONS OF MAIN PIERS



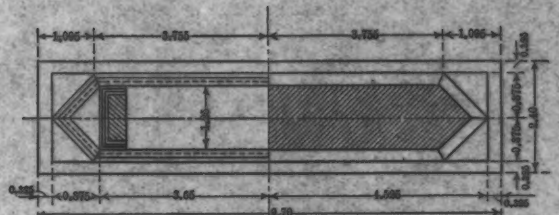
ELEVATION SECTION



SECTION M-M HORIZONTAL SECTION AT H-H SECTION N-N



SECTION OF PIER SECTION K-K ELEVATION SECTION L-L



SECTION S-S SECTION R-R
INTERMEDIATE PIERS

PICAZA BRIDGE
DETAILS OF
MAIN AND INTERMEDIATE
PIERS

and it may be of interest to note that this mosaic gives a very effective and beautiful contrast to the concrete.

The ornamental brackets on the side spans were cast in Buenos Aires on a foundation of expanded metal. They are hollow, and very carefully hammer-dressed to resemble cut stone. They were tied in place to the sidewalk reinforcement through tubes, purposely left in the concrete, which were afterward grouted from above and sealed.

Surface Finish.—All masonry above the cornice level was plastered with 1:2 sand mortar, and, after hardening, was hammer-dressed to give the appearance of cut stone. The cornices were plastered with the same mortar but combed with a fine steel comb, giving an imitation of cut stone. This method is used effectively in the Argentine on nearly all buildings of architectural merit.

Below the cornice level a rough granolithic surface was secured by first making longitudinal grooves with cement mortar and, when these were hard, plastering between the grooves with granite chips, and cleaning the surface subsequently. This produces a granite surface which reflects the light and is broken by smooth horizontal lines—a combination which contrasts well with the smooth pylon columns.

The exposed sides of the side spans were left rough and scrubbed with wire brushes; then a coarse sand mortar was rubbed on the rough surface, using small grindstones for that purpose; two coats were applied, the second on the first before it had set. In this way a hard impermeable rough surface was obtained, which, at a distance of 3 m., can hardly be distinguished from the hammer-dressed portions. It may be pertinent to call attention to the fact that a surface of uniform color and texture was desirable because the necessary use of different cements on different spans had produced a variety of shades, not at all pleasing, and this finish corrected this defect.

The upper and lower surfaces of the arches were finished smooth. The hangers and tension members were plastered on the picked concrete, in order to obtain the curves of the design. The sides of the arches were well picked and then plastered with fine granite chippings, and wire-brushed 12 hours afterward to remove the cement coating. A fine and very satisfactory granolithic surface was obtained. The under surfaces were simply washed with cement to produce a uniform tone.

In all cases the objects have been to obtain a uniform tone, to break up the light, and, at the same time, to avoid monotony in the appearance of the different classes of surface. It may be said that these objects have been achieved, although, as the granite used is of exactly the same color as the cement, far greater and more pleasing contrasts could have been obtained with an aggregate of a different color.

The appearance of the bridge is pleasing, and it is of interest to note that although a near view gives the idea of massiveness, at a distance of 1 km. (3 280 ft.), the impression conveyed is that of extreme lightness.

General Observations.—After a year of sun, frost, and floods, it has been noticed that there are no structural cracks in the work. A few have appeared in the mortar plastering, but always near or at expansion joints or bearings of the concrete beams, and these, in all cases, have been due to the plastering of the brickwork not having been isolated completely (by cutting) from the movable beam surfaces.

The effect of variations in temperature on the length of the beams is very noticeable. On a cold night a movement of 1 cm. (0.39 in.) is easily seen on the lateral spans, and the central span contracts 2 cm. (0.79 in.). The range of temperature is from 0° to 35° cent., but the most sudden changes are in the early spring and late fall when freezing temperature at daybreak is followed by high temperature at noon, or within 5 hours.

Progress According to Schedule.—A schedule of work was made and adhered to as closely as possible, but there were many causes for delay. For instance, there was a compulsory slow-down of 2 months caused by a general strike of the railroad locomotive drivers; materials were delayed in arrival, the rate of work was lessened, and, in fact, all work was affected. Another cause of delay was in the sinking of the cylinders, which was not as rapid as expected, owing to the difficulties mentioned. This was an important feature because the completion of the central piers controlled the time of finishing the spans resting on them. There were also minor delays caused by modifications in design, lack of paving materials (which were imported from Sweden and were late in arrival), and for 3 months at the beginning of the work there were not enough competent carpenters for form work. The work began in January, 1912, and

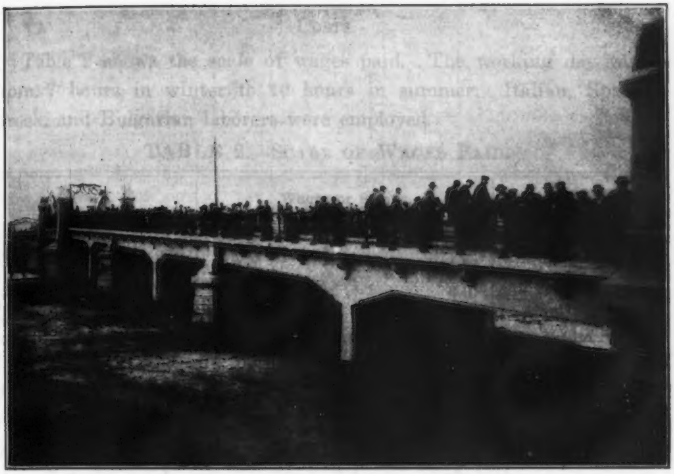


FIG. 5.—GENERAL VIEW OF PICAZA BRIDGE, AFTER OPENING TO TRAFFIC.

FIG. 6.—GENERAL VIEW OF PICAZA BRIDGE.

traffic was passing over the bridge in December of the same year, although all the surface treatment had not then been completed.

Costs.

Table 2 shows the scale of wages paid. The working day varied from 7 hours in winter to 10 hours in summer. Italian, Spanish, Greek, and Bulgarian laborers were employed.

TABLE 2.—Scale of Wages Paid.

Duty.	Wages or Salary.	Equivalent American Currency.
General foreman	$220.00 and bonus, average $280.00	$123.00
Concrete foreman	125.00 " " " 150.00	66.00
Concrete laborers	3.20 " " " 4.50	1.98
Cylinder laborers	4.00 " " " 5.50	2.42
Carpenters	6 to $7 " " " 8.00	3.52
Masons	5 to 7 " " " 8.00	3.52
Time and storekeeper	120.00 120.00	52.80

Tables 3, 4, 5, and 6, showing the costs, are self-explanatory. Owing to the fact that the costs of materials, transportation, and overhead charges not incurred on the work itself, were not kept by the writer, he regrets that he cannot detail them as he has the labor costs, but the totals are correct, and he trusts that they may give an idea of the cost of the work under given conditions in a distant country. To an American engineer, the proportion of money spent on purely esthetic grounds in a prairie region may seem to be too great, but such seems to be the custom of the country, and perhaps, eventually, on broad grounds, it may be regarded as proper for permanent structures of this type.

TABLE 3.—Costs of Labor and Materials on Picaza Bridge.

Class of work.	Cost.	Units.	Cost per Unit.		Observations.
			Pesos.	U.S. gold dollars.	
Excavation, foundations	791.16	1 023.3 m.	0.77	0.33	Average depth, 1 m.; average haul, 150 m.
Excavation, sinking cylinders	2 015.08	283 m.	7.12	3.03	To 9 m. below surface. Haul, 200 m. includes pumping.
Cylinders, sand filling	71.71	50 m.	1.43	0.61	Including transport and loading at 200 m.
Concrete:—1 : 3 : 5					
Foundations	984.03	301 m.	3.27	1.39	Mix and place, including some reinforcement.
Cylinders	886.17	174 m.	5.09	2.16	Including forms, reinforcement and plastering.

TABLE 3.—(*Continued.*)

Class of work.	Cost.	Units.	Cost per Unit.		Observations.
			Pesos.	U.S. gold dollars.	
Reinforced Concrete :—					
1 : 2 : 2 and 1 : 2 : 1½					1 : 2 : 1½ only in arches and hangers, 48.3 m.
Reinforcement, prepare.	670.37	110 tons.	6.09	2.59	Bending and transporting from shop to place.
" place...	980.75	110 tons.	8.91	3.79	Place and tie.
Forms, erect and take down	5 751.12	732 cu. m.	7.86	3.34	Two sets used three times for lateral spans, one set for central span, cost 50% more.
Concrete, mix and place	2 964.71	732 cu. m.	4.05	1.73	Central span = 125.3 m.
Masonry :					
Brickwork, 1 : 3 cement mortar	4 545.19	1 009.3 m.	4.50	1.91	250 tons of rubble utilized in masonry, including piers, abutments, and ornamental pillars.
Plastering on Brickwork :					
Imitation granite	1 349.96	529.2 m.	2.55	1.08	Granite chippings brushed, broken horizontally by smooth channeling.
Imitation fine cut stone...	2 180.50	623 m.	3.50	1.49	Plastered 1 : 2 fine sand and combed with fine comb.
Rustic	191.10	273 m.	0.70	0.30	Rough cast plastering.
Finishing Concrete Surfaces :					
Plastered and rubbed	986.64	793 m.	1.24	0.53	Plastered and rubbed with grind-stone, 2 coats.
Fine plastering	1 627.50	465 m.	3.50	1.49	Plastered 1 : 2 fine sand and combed with fine comb.
Imitation granite	267.00	89 m.	3.00	1.27	Granite chippings brushed, broken horizontally by smooth channeling.
Cement wash	338.40	1 692 m.	0.20	0.09	Applied with brush, 2 coats.
Hammer-dressing Mortar Surfaces	584.48	244 m.	2.39	1.02	On plaster 80% ; on narrow concrete bands, 20 per cent.
Modifications :					
Tie-wall pylons	481.75	161 m.	29.92	12.72	Between columns, including picking sides of columns and reinforcement.
Transverse arches	119.63	3 m.	39.87	16.94	Central span wind bracing, including forms, reinforcement and plastering.
Ornamental Castings :					
Central span	269.40	12 No.	22.45	9.54	Place negatives, grout with 1 : 2 mortar, and finish surfaces.
Lateral-brackets	335.76	84 No.	3.99	1.70	Place, grout, and plaster bevels, etc.
Steel Hand-railing :					
Rectify	212.00	380.1 m.	0.56	0.24	Made in Buenos Aires, received bent.
Place and rivet	604.76	380.1 m.	1.59	0.68	Including bracing while concreting sidewalks.
Paint, 3 coats	484.79	271 tons.	1.79	0.76	
Pavement :					
Water-proofing concrete..	396.00	990.2 m.	0.40	0.17	Coal-tar and sand applied hot and ironed.
Surface and sub-surface drains	76.42	36	2.12	0.90	16 surface cast-iron inlets ; 16 sub-surface 2½-in. pipes, and 4 inlets, cast iron and brick, on abutment areas.

TABLE 3.—(*Continued.*)

Class of work.	Cost.	Units.	Cost per Unit,		Observations.
			Pesos.	U. S. gold dollars.	
Asphalt expansion joints.	85.43	6	14.24	6.05	Each about 3.50 sq. in. and 0.05 in. thick (between spans).
Swedish block paving.....	2 080.00	1 040.2 m.	2.00	0.85	10-cm. cubes placed in fan-shaped mosaic and joints tarred.
Granite curbing.....	608.33	426.1 m.	1.43	0.61	$286.45 cutting to required section.
Concrete base (abutments)	72.49	14.7	4.93	2.10	To allow uniform settlement of pavement.
Sidewalk (asphalt tiles)..	273.26	278.2 m.	0.98	0.42	($3 591.93 pavement.)
Steel Reinforcing Arch:					
Central span, including bearings.....	1 186.52	24 tons.	49.44	21.01	Including falsework and adjusting.
Structural steel in expansion joints.....	81.95	.....	.....	.....	To reinforce brickwork.
Wash and clean broken stone	523.06	1 000 tons.	0.52	0.22	Very dirty from being deposited 2 years in exposed place.
Handling materials on work	1 238.90	5 540 tons.	0.22	0.09	From stone piles, including loading and transporting 150 m.
Unloading from carts....	4 561.60	6 900	0.66	0.28	Materials and plant 60% done 1 year before work began, and without supervision.
Back-fill and tamping.....	807.80	.....	.....	.....	In 12-in. layers, and wet.
Concrete trestle — erect and take down.....	1 399.17	Twice.	699.58	297.32	Trestle 100 m. long, average height 6 m., erected and taken down twice.
Repairs to plant and equipment	893.41	.....	.....	.....	Tools, derrick, etc.
Camp:					
Installation	1 338.05	.....	.....	.....	Including $735 prior to commencing work.
Removal	436.66	.....	.....	.....	
Drainage.....	700.70	.....	.....	.....	
Deviation of highway.....	709.55	.....	.....	.....	To maintain traffic while building bridge.
Ramps and Level Crossings:					
Formation of.....	6 761.16	11 542.6	0.585	0.25	10 293.3 m. ramps, level crossings 1 249.3 m.
Sodding.....	342.11	.....	.....	.....	
Fencing ramps.....	386.13	.....	.....	.....	
Testing bridge.....	154.88	.....	.....	.....	
Total labor.....	54 307.54	.....	.....	.....	
Administration:					
On work.....	9 262.69	.....	.....	.....	Cook, inspector, etc. Of this, only $5 678 belongs to construction period, the $3 131 was due to delay in beginning work.
General overhead charges from Dolores office.....	8 787.03				
Total administration..	18 049.72				

TABLE 4.—Approximate Cost of Materials, Picaza Bridge.

Materials.	Units.	Unit price.	Cost.	Observations.
Cement	672 tons.	$61.24	$41 153	As these costs of material are posted monthly by the Dolores office, the unit prices given are approximate averages. All freight handling charges at Buenos Aires and Dolores are included.
Sand	1 680 "	4.90	8 232	
Broken stone and screenings	1 914 "	6.10	11 675	
Rubble stone	250 "	4.20	1 050	
Steel reinforcing bars, round	110 "	118.00	12 980	
Steel, structural in arch	24 "	407.83	9 788	
Steel, ornamental railing	27 "	618.52	16 700	
Brick	360 000	23.00	8 280	
Swedish granite paving sets	1 040.2 m.	7.00	7 280	
Curbing	4 461 m.	7.00	3 122	
Asphalt tiles for sidewalk	4 m.	400.00	1 600	
Ornamental brackets	83	8.29	688	
Ornaments (negatives)	4	40.00	160	
Wire, bearing plates, etc.	.....	.....	4 500	
Carting	6 900 tons.	2.00	13 800	
Total	.....	.....	$141 008	

TABLE 5.—Résumé of Cost of Picaza Bridge.

Labor		$54 307
Materials (transportation, etc., on work)		141 008
Plant: Depreciation	$5 000	
Wood, coal, etc., consumed	6 000	11 000
Administration and supervision		18 049
Grand total		$224 364

TABLE 6.—Costs of Picaza Bridge per Unit of Length and Surface.

	Length, Over All.		Paved Area, Over All.	
	Including abutments.	Excluding abutments.	Including abutments.	Excluding abutments.
Quantities	210 lin. m.	197.80 lin. m.	1 494 sq. m.	1 424 sq. m.
Pesos per linear meter	1 068.40	1 134.30	.....	.....
Pesos per linear foot	325.63	345.72	.....	.....
U. S. gold dollars per linear foot	$138.39	$146.93	.....	.....
Pesos per square meter	.....	.....	150.10	157.50
Pesos per square foot	.....	.....	13.95	14.64
U. S. gold dollars per square foot	.....	.....	$5.93	$6.22

Total volume of masonry, including concrete = 2 215.3 cu. m. = 2 899 cu. yd.
Area, over all (excluding abutments) ..... = 1 424 sq. m. = 15 322 sq. ft.
Equivalent to 0.19 cu. yd. per sq. ft.
$1.00 U. S. currency = 2.354 pesos.
1 peso = $0.425 U. S. currency.

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Paper No. 1354

PEARL HARBOR DRY DOCK*

By H. R. Stanford,† M. Am. Soc. C. E.

With Discussion by Messrs. Charles Evan Fowler, Harrison S. Taft, W. F. Frear, P. L. Reed, Frederic R. Harris, Leonard M. Cox, W. H. Pretty, J. R. Baterden, Herbert E. Bellamy, B. C. Laws, Edward Box, and H. R. Stanford.

Synopsis.

The object of this paper is to inform those interested in engineering matters regarding the peculiar difficulties and problems which the construction of the Pearl Harbor Dry Dock has involved.

The paper contains a topographical and geological description of the location which was chosen for the dock, a review of the Congressional authorizations and appropriations for the work, a brief description of the plans and methods of construction originally adopted, an outline of the changes which were made in dimensions and details as the work proceeded, and a description of the field troubles which were experienced and of the experiments and tests conducted to determine the best means for their correction.

The paper concludes with a description of the plans and methods which were finally adopted and under which the work is now proceeding.

A paper on the Pearl Harbor Dry Dock at this time may on first thought be considered premature, inasmuch as little progress has been

* Presented at the meeting of September 1st, 1915.

† Corps of Civil Engineers, U. S. Navy; Chief of Bureau of Yards and Docks, Navy Department.

made toward actual completion, but a description of the work from its inception to date seems to be justified, in view of the numerous and peculiar construction difficulties which have been experienced, its military importance, its interest as an engineering project, and the recent resumption of operations on radically modified plans. It must be borne in mind, while reviewing the history of this project, that we now have the inestimable advantage of "hindsight" for our guidance, and that the problems involved should be weighed accordingly.

Location.

Pearl Harbor is in the southern part of the Island of Oahu, of the Hawaiian group, and is a body of water of irregular shape, formed by several lochs or arms of the sea. The depth of the water in the harbor generally varies from 35 to 50 ft., and these depths are well maintained to the shore line. The land surrounding the harbor is comparatively level, at an elevation of about 12 ft. above mean low tide. The tidal movements average about 1 ft. 2⅜ in. The channel connecting the harbor with the sea was naturally shallow and rather crooked, but, as a result of dredging operations completed in 1912, it now has a least depth of 35 ft. over an entrance width of 600 ft. and a minimum width of 500 ft. within the entrance, with curves eased to permit of the safe passage of battleships. The dredging operations required the removal of 4 645 000 cu. yd. of material, throughout a channel 5 miles long, at a cost of $3 296 000, and are of engineering interest because of the successful manner in which the work was conducted over the entrance bar, where there was a heavy sea swell at all times.

The area included in the Pearl Harbor Station was acquired in February, 1904, by the Bureau of Equipment of the United States Navy Department, under its authority to purchase land required for the storage of coal.

The Act of Congress, approved May 13th, 1908, making appropriations for the naval service, includes an item reading:

"The Secretary of the Navy is hereby authorized and directed to establish a naval station at Pearl Harbor, Hawaii, on the site heretofore acquired for that purpose; and to erect thereat all the necessary machine shops, storehouses, coal sheds, and other necessary buildings, and to build thereat one graving dry dock capable of receiving the

largest vessels of the Navy, at a cost not to exceed two million dollars for said dry dock."

The location of the naval station is especially favorable because of the proximity of natural deep water to the large areas of low-lying level land which are included within the boundaries of the property. The ground conditions are fairly good for the support of heavy concentrated loads, but sub-surface operations, which require the exclusion of ground-water, are costly because of the porous character of the soil. The railroad and highway facilities between Honolulu and the station are excellent, and the existence of the large and convenient commercial center is of inestimable value to the station as a base for labor and supplies.

The first general outline plan for the development of the Pearl Harbor Naval Station, following its Congressional authorization, was submitted in a report dated July 22d, 1908, by a Board of which Rear-Admiral Seaton Schroeder, U. S. N., was senior member. The location for the dock shown on this plan was slightly changed in its axial and transverse position, and the design was directed for a dock having a length of 1 140 ft. with an intermediate caisson to provide two chambers, 564 and 536 ft. long, respectively, in accordance with the recommendation of C. W. Parks, M. Am. Soc. C. E., Civil Engineer, U. S. N., and the Bureau of Yards and Docks, by authority of the Navy Department dated November 19th, 1908. The location for the dock was again changed by the Navy Department on March 19th, 1909, by moving the dock on its axis a distance of 200 ft. inshore, as a result of the great apprehension expressed by bidders regarding the dangers and difficulties which would attend the building and maintaining of a coffer-dam in the deep water in front of the dock entrance.

Geological.

The Island of Oahu is of volcanic origin, and consists of an elongated hilly or mountainous section, about 45 miles in length, lying in a southeast and northwest direction, with a surrounding fringe of low land formed by accretions of coral and volcanic origin. Pearl Harbor is on the south shore of the island, within the fringe formation, and is about 8 miles from Honolulu.

The geological conditions at the site of the dock are of very great importance in the consideration of the difficulties which have been

experienced in the construction of the work, and therefore warrant a careful description. The surface soil, to depths exceeding those reached by any part of the dock, is either volcanic or organic sediment. The volcanic sediment or tuff beds result from the intermixture of ashes and volcanic stones fallen from the air, with pebbles and sands made of hard lava washed in from the island valleys, the combination being, in some cases, re-arranged by water into beds of sand, or mud, or gravel. The material of organic origin consists of coral skeletons or shells ranging from hard reef, as originally deposited by coral animals, to broken and disintegrated material heaped together by the waves or assorted into gravels, sands, and muds, or intermixed with volcanic materials. There is no regular stratification of the material, and its lack of uniformity and its wide variation between the limits of hard and soft may be judged from the penetration records of various pairs of adjacent piles (piles in any one pair not more than 4 ft. apart) which were driven to refusal in the bottom of the dock excavation, as follows: 9 and 23 ft.; 5 and 26 ft.; 6 and 25 ft.; 3 and 24 ft.; 22 and 37 ft.; 11 and 35 ft.; 15 and 43 ft.; 5 and 32 ft.; 13 and 21 ft.; 11 and 22 ft.; 10 and 27 ft.; and 14 and 26 ft.

Described in detail, the soil comprises the following elements:

Rock Types:

Volcanic Tuff.—A hardened volcanic ash.

Volcanic Grit.—A hardened coarse volcanic sand.

Volcanic Agglomerate.—A hardened volcanic gravel made of angular fragments formed by explosion.

Coral Rock.—Material of an old reef still in place.

Soft Deposits:

Volcanic Mud.—A fine-grained aggregate of lava minerals formed by the wash of river or sea.

Volcanic Sand.—An aggregate of volcanic minerals, generally dark-colored, formed by wash of river or sea, and assorted to grains of fairly uniform size. The grains will be more or less rounded, according to the distance they have traveled and the amount of wear to which they have been subjected.

Volcanic Gravel.—Like volcanic sand, but coarser and of variable sizes. A true volcanic gravel containing pebbles of basaltic lava larger than the fragments in the Pearl Harbor tuffs, and

occurring in their vicinity, is presumptive evidence of the existence of a river channel from the hills, as it would be impossible for shore currents of the ocean to carry pebbles of ledge rock from any point on the shore of the island where such rock is exposed to the waves, along the coast and into the tortuous channels of Pearl Harbor without reducing them to sand or mud.

Coral Mud.—A fine-grained, white, yellow, or buff mud, made of crushed coral and shells.

Coral Sand.—Assorted grains of coral and shells of fairly uniform size.

Coral Gravel.—Washed fragments of coral and shells, more or less rounded, and more or less mixed with sand.

Mixed Deposits:

Tuff-Coral Mud.—A fine-grained aggregate of particles of tuff and of tuff minerals, mixed with particles of coral and shells.

Tuff-Coral Sand.—An aggregate of volcanic sand and coral sand mixed.

Tuff-Coral Gravels.—An aggregate of volcanic gravels and coral gravel mixed.

Basalt-Coral Gravel.—An aggregate of coral gravel and gravel made of partly rounded fragments of basaltic lava.

The softer classes of the foregoing materials become a mud or sand when soaked with water, but the solid coral and tuff rocks maintain their integrity. The volcanic deposits effervesce in dilute hydrochloric acid only in such proportion as they contain infiltrated carbonate vein material, whereas the corals, coral sand, and coral mud—which are composed almost wholly of carbonate of lime—effervesce very strongly.

An examination of the side slopes of the practically completed excavation for the dock, and of boring records, indicates a close resemblance between the underlying materials at the dock site and formations exposed in road cuts in the vicinity, except that the former are thinner-bedded and more uniformly horizontal, because of their greater distance from their source. Figs. 1 and 2 are photographs with notations which illustrate the principal formations. Borings for Artesian wells in the Pearl Harbor district reveal the great

thickness of these coral and volcanic deposits below sea level, thus clearly indicating a history of gradual submergence and explosive volcanic activities, with occasional uplifts of the deposits.

Design.

The usual modern dry dock of the graving type is essentially a masonry prism resting in an earth excavation. Inward earth and water pressure against the side-walls and upward hydrostatic pressure on the bottom are the principal forces which the prism must be designed to resist. The dry dock prism thus economically becomes a gravity arch structure, made of concrete, which depends on the weight of its material and arch action for stability. Water-tightness is incidentally obtained by the proper proportioning of the concrete ingredients. The main problem to be solved in the construction of a dock, therefore, depends on obtaining impervious monolithic and homogeneous concrete of permanent character, and having maximum unit weight combined with an ordinary degree of strength and hardness.

These comparatively simple requirements formed the basis of the design which was originally adopted for the Pearl Harbor dock. They assumed that concrete could be placed under the favorable dry and open conditions which usually obtain, and that earth friction on the sides of the prism, combined with the natural resistance of the underlying soil to infiltration, would warrant a reduction in the concrete in the prism to a quantity which would weigh somewhat less than the equivalent of a full upward hydrostatic pressure on the bottom.

As the work progressed, it became apparent that the usual open coffer-dam method of construction could not be followed, because of the porous and water-bearing condition of the soil. Arrangements were then made whereby foundation piling with heads buried in the bottom concrete was provided, and the contractor was permitted to place not to exceed the lower 8 ft. of bottom concrete by the under-water method. There was no provision, however, for any material enriching of the mix to offset losses and difficulties which the under-water method involved. The futility of this procedure was demonstrated by the defects in the 6 284 cu. yd. of concrete which had been placed to an average thickness of 6.52 ft. over a section of 180

FIG. 1.—ROADSIDE CUTTING, SHOWING MATERIALS FOUND IN EXCAVATION FOR PEARL HARBOR DRY DOCK.

FIG. 2.—ROADSIDE CUTTING, SHOWING MATERIALS FOUND IN EXCAVATION FOR PEARL HARBOR DRY DOCK.

lin. ft. at the inner end of the dock site, and which was partly exposed when the surrounding coffer-dam was unwatered to a depth of 37.3 ft.

The next modification in the work consisted in changing the ingredients in the under-water concrete, making the mix 1 part cement, 2 parts sand and 3½ parts broken stone, and in which one-third of the sand was silicious, from the mainland, and two-thirds were screenings from crushed lava rock. This richer mixture was placed to an average thickness of 6.75 ft. over a section of dock 240 ft. long adjacent to the inner section in which the lean concrete had been used. The under-water concrete in this section was placed in forms approximately 12 ft. square, and it was arranged that adjacent blocks should be finished with upper surfaces in planes separated by about 12 in., in order that an effective bond could be made between this lower concrete and that which was to be placed later when completing the floor. The collapse on February 17th, 1913, occurred in this second section, but, as all records agree that the concrete was of excellent quality, it may be asserted that the attempt to obtain satisfactory concrete in deep water by the tremie method was successful, and that the collapse was chargeable to other causes.

By this time it was recognized that local conditions demanded, not only unusual and expensive methods of construction, but also that the concrete should be exceptionally rich in quality with increased quantity, and that extensive metal reinforcements were necessary in the bottom to provide effectively for irregularities in the supporting power of the underlying soil. The recent change in contract recognizes all these needs; it provides a structure which is heavier than the water displaced; practically eliminates all concrete which must be placed under water; permits of thorough inspection; introduces substantial steelwork to relieve any probable tensile stresses in concrete; and provides for the independent settlement of seventeen practically equal portions of the dock before combining them into a single unit, thereby insuring the completed dock against settlement cracks and leakage.

Appropriations and Contracts.

Specific Congressional authorization is necessary before undertaking the construction of any public works improvement required for the Navy. The dry dock for Pearl Harbor was originally au-

thorized in an Act, approved May 13th, 1908, which fixed the limit of cost at $2 000 000. This limit was increased to $2 700 000 and then to $3 486 500 by Acts approved June 24th, 1910, and August 22d, 1912, in order that the dock could be made 1 000 ft. long and 110 ft. wide, to accord with the dimensions of the Panama Canal locks. The limit of cost was again increased by an Act, approved June 30th, 1914, and made $4 986 500, to provide for radical changes in plans to obtain a more substantial dock and reduce building hazards.

Contracts for naval public works always contain a provision whereby the Government reserves the right to make such changes in the contract, plans, and specifications as may be deemed necessary or advisable. The actual modification of a contract to cover a change of magnitude is accomplished with a document called a "Supplemental Agreement", which is acknowledged by the surety, accompanied with additional bond, and finally executed in the same manner as the original contract. Contracts and supplemental agreements for the Pearl Harbor Dry Dock in accordance with the Congressional authorizations previously cited, have been made as follows:

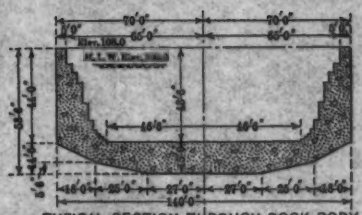
Original contract, dated July 22d, 1909, awarded to the San Francisco Bridge Company of San Francisco, Cal., provided for a dock to receive a vessel having dimensions not to exceed: length, 581 ft. 0 in.; beam, 105 ft. 4 in.; and draft, 31 ft. 8⅜ in., with clearance of 3 ft. 0 in. over the entrance sill, to cost $1 760 000.

Supplemental Agreement "A", dated June 27th, 1910, enlarged the dock to receive a vessel having dimensions not to exceed: length, 792 ft. 0 in., and beam, 113 ft. 4 in., and added $544 000 to the original contract cost.

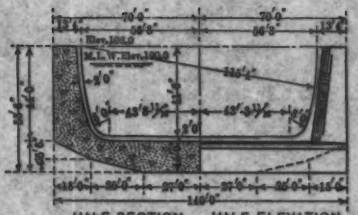
Supplemental Agreement "B", dated August 5th, 1911, provided foundation piles for the dock, with an increase in contract cost of $150 643.22.

Supplemental Agreement "C", dated January 2d, 1913, enlarged the dock to receive a vessel 1 000 ft. long, provided for enriching the bottom concrete, and added $711 842.50 to the contract cost.

Supplemental Agreement "D", dated November 19th, 1914, made radical changes in the details of the plans and specifications, without materially changing the dimensions of the dock, and added $1 261 350 to the contract cost. This addition, together with sums added on

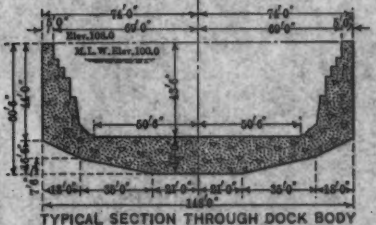


TYPICAL SECTION THROUGH DOCK BODY

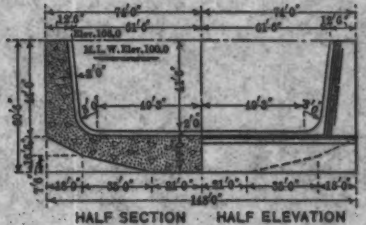


HALF SECTION THROUGH SILL — HALF ELEVATION OF ENTRANCE

Design covered by original Contract Dated July 22d, 1909

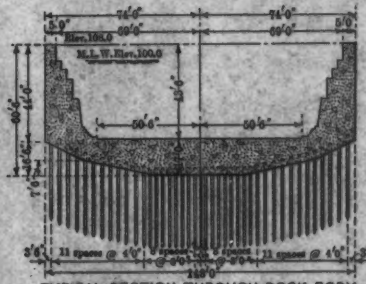


TYPICAL SECTION THROUGH DOCK BODY

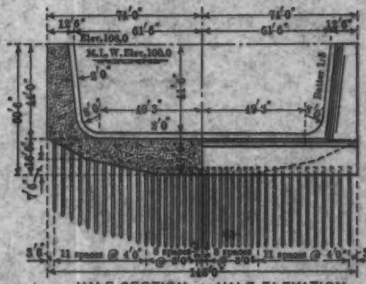


HALF SECTION THROUGH SILL — HALF ELEVATION OF ENTRANCE

Design after supplemental agreement Dated June 27th, 1910
Piling as shown below added by S.A. Dated, August 15th 1911

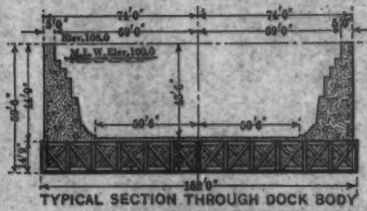


TYPICAL SECTION THROUGH DOCK BODY

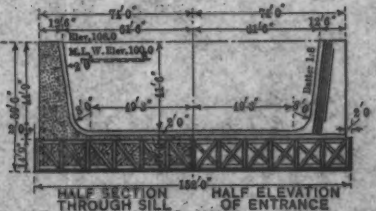


HALF SECTION THROUGH SILL — HALF ELEVATION OF ENTRANCE

Design after supplemental agreement Dated January 2nd, 1913

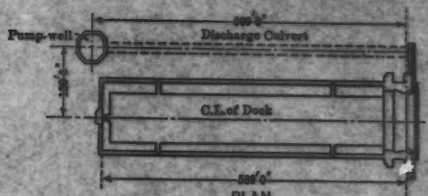


TYPICAL SECTION THROUGH DOCK BODY



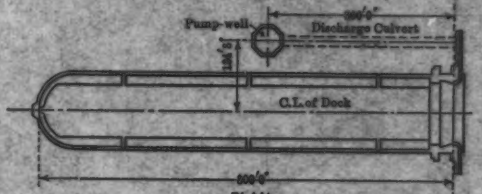
HALF SECTION THROUGH SILL — HALF ELEVATION OF ENTRANCE

Design after supplemental agreement Dated November 19th, 1914



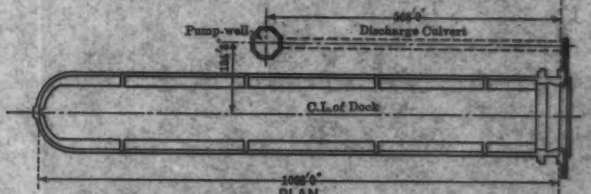
PLAN

Design covered by original Contract Dated July 22d, 1909



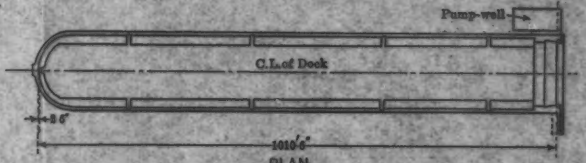
PLAN

Design after supplemental agreement Dated June 27th, 1910 and August 15th, 1911



PLAN

Design after supplemental agreement Dated January 2nd, 1913



PLAN

Design after supplemental agreement Dated November 19th, 1914

DIAGRAMS SHOWING ENLARGEMENTS AND CHANGES IN THE DOCK

account of various minor changes, makes the total obligation under the contracts amount to $4 442 115.43.

Plate V shows the general outline dimensions of the dock as it has developed under the foregoing changes.

Supplemental Agreement "C" is considered of particular interest to the engineer because of the special provision which it contains to place full responsibility more definitely on the contractor for the completion of the dock, and also because of the relation which this condition created between the contractor and the Government's engineer in charge at the time of the collapse of the work on February 17th, 1913, and further because of its bearing on the negotiations preliminary to resuming work on radically changed plans subsequent to the collapse.

Prior to the execution of this supplemental agreement there had been a conference between the Chief of the Bureau and the contractors, during which the latter were informed that inasmuch as it was proposed to modify the contract to provide a dock of increased length, which would practically obligate the entire amount authorized for the construction of the dock, it would be necessary for the contractor to agree definitely to complete the work without any further increase in contract price, regardless of any mishaps or accidents which might subsequently be experienced. It was explained to the contractor that a primary reason for entering into contract for the construction of the dock was to insure the completion of the work within the amount authorized by Congress. In order to avoid interference with the operations of the contractors, which would tend to relieve them of their responsibility, it was arranged that the greatest possible latitude would be given in conducting the work; in other words, that it would be left largely with them to determine such details as the quantity of water which should be added in mixing concrete as affecting its consistency, the dimensions of the forms into which tremie concrete should be placed, the manipulation of the tremie, etc. In accordance with this understanding regarding responsibility, the following paragraph was included in the supplemental agreement forming a part of the contract:

"That it is specially understood and agreed that the party of the first part may elect any adequate methods or details of methods of construction to be employed in the construction of said dry dock and its appurtenances, and that the said party of the first part assumes full and complete responsibility for the adequacy of the materials specified and of the methods and details of methods of construction to

be employed and for the successful and satisfactory completion of the entire work in all its parts, and agrees to make good at its own expense any defect or inadequacy of the materials as herein or elsewhere specified and any defective or unsatisfactory work: *Provided,* That all materials and workmanship and the adequacy of all methods and details of methods of construction shall, as contemplated by the contract, be subject to the approval of the party of the second part. And it is further specially understood and agreed that all items of extras, damages, claims, time allowances, or other matters that have been or might be the subject of differences of opinion or dispute are merged in this agreement and satisfied and extinguished by the consideration herein stated and that no claim shall hereafter be presented or entertained on account of anything connected with the construction of said dry dock and appurtenances occurring or existing prior to the date of this agreement."

Having in mind this special provision of the contract and the circumstances which led to its adoption, let us consider the position of the officer in charge representing the Government during the several days immediately preceding the collapse of the work on February 17th, 1913. During this period the cribbing and sheet-piling protecting the section which was being unwatered had been showing increasing evidence of distress, up to the time of the collapse, when the bottom was almost exposed and in condition for the placing of concrete, each yard of which would have reduced the building hazard. It should also be noted that the conditions in this crib at this time were not very different from those which had existed several months before when Crib No. 1 was unwatered to practically the same depth without disastrous results.

The officer in charge, an experienced engineer, was fully cognizant of the extreme hazard of the situation prior to the collapse, but, knowing the responsibility on the contractor for the safe conduct and completion of the work, any interference on his part, based solely on his individual judgment, would probably have involved serious responsibility on the Government in defending his action as affecting the period required for the execution of the work, and also the contractors' expenditures. The great distance of the work from the Bureau and from other engineers with whom the officer in charge might have consulted, combined with the brief period during which the weakness of the coffer-dam was in evidence, together with knowledge of the contract provisions regarding the contractors' responsibility, resulted in placing the officer in charge in a very difficult position. Now that

the occurrence with all its details is a matter of history, it is very easy to conclude that some other course of procedure than that actually followed might have been better and proper; but, under the circumstances, it is doubtful if others would have done differently.

The negotiations between the Government and the contractors following the collapse of the work, were based on the claim of the contractors that the contract plans were insufficient to provide a safe and satisfactory dock; no claim was made on account of losses sustained. The contractors claimed, however, that it was the plans which were at fault, and that the Government was responsible for the plans; they further represented that any radical change in plans could not be considered as proper within the meaning of the paragraph on changes as contained in the contract, and that if such radical changes were desired it would be optional with the contractors as to whether they should agree to them. It was undoubtedly desirable that the contractors should proceed with the work in view of the valuable experience which they had acquired and in order to utilize the plant which had been assembled; the contractors on their part were anxious to proceed because of their business and professional standing. The Department of Justice, after a review of the contract, states, in an opinion given January 5th, 1914,

"Of many building contracts, I have never seen one that so persistently declared for full responsibility on the contractor during the course of construction."

In view of the foregoing conditions, the Bureau of Yards and Docks, which is charged with the design and construction of naval public works, felt constrained to withhold any proposal to adopt new methods or plans for the work until after the Secretary of the Navy should decide in favor of a stronger, more reliable, and more costly structure. This decision was reached by the Secretary early in April, 1914, after obtaining the advice of the late Alfred Noble, Past-President, Am. Soc. C. E., and various members of the Corps of Civil Engineers of the Navy; the general design for the dock which was subsequently adopted was then presented by the Bureau during the same month.

Field Operations.

Excavation.—Excavation for the dock was made with a dipper-dredge to a depth of from 35 to 38 ft. below low-tide level, and completed

to the required depth with a hydraulic dredge fitted with a rotary cutter. Dredging was begun in October, 1909, and practically completed in December, 1910. The material was loosened in advance of dredging by blasting, which required drilling 3-in. holes, from 6 to 10 ft. apart transversely, and somewhat farther apart longitudinally with the dock; about 50 lb. of 40% gelatine was used for each hole.

Coffer-Dam.—The open coffer-dam method of construction was adopted by the contractor when work was first started, and was followed up to the time that operations were temporarily suspended in February, 1913. The distance between the side-walls of the coffer-dam was about 148 ft., which was sufficient to include the full width section of the dock. The length of the coffer-dam was divided into sections by transverse bulkheads. The excavation required within a section of coffer-dam was completed in advance of building the coffer-dam. Interior cribwork was used for bracing the coffer-dam walls, as was done in constructing the dock at Mare Island, California, where the same contractors' superintendent was in charge. The walls of the coffer-dam were of 12-in. square timbers; two pieces of timber were first bolted together to form a unit, after which beveled-edged strips of 3-in. stock were secured to opposite edges of each unit, two strips on one edge and one strip on the opposite edge, the bevels and spacing of the strips being arranged so as to obtain a dove-tailed joint connecting adjacent units.

The cribwork comprised essentially a system of six horizontal tiers; each tier was of transverse and longitudinal timbers 12 ft. from center to center; a vertical timber standing on a concrete footing piece at the bottom of the excavation bound together the several tiers at each panel point of each bent; the upper tier was about 3 ft. above low-tide level, and the lower tiers were at intervals of 11.5, 10.1, 9.0, 8.0, and 8.0 ft., respectively; the first, second, and sixth tiers and the vertical members were of 12-in. square timber, with an 18 by 22-in. wale for the second tier; the third tier was of 14-in. square timber, with two 12 by 24-in. wales; the fifth tier was of 16-in. square timber with two 14 by 24-in. wales; the fourth and fifth tiers were braced together at each bent by intermediate transverse bracing to form Howe trusses; there was one line of diagonal bracing for each bent. Figs. 3 and 4 are photographs showing the coffer-dam and crib.

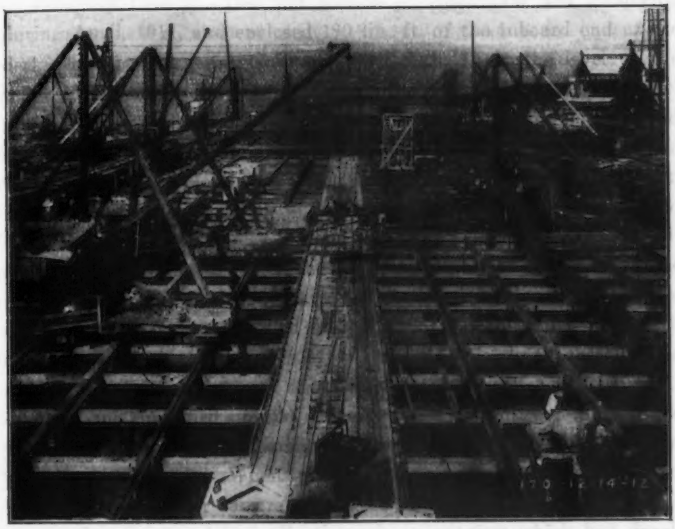
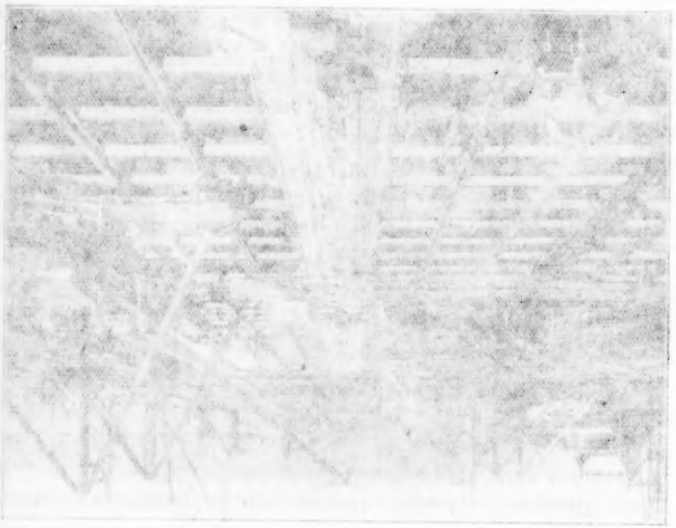


FIG. 3.—COFFER-DAM, PEARL HARBOR DRY DOCK.

FIG. 4.—CRIBWORK IN COFFER-DAM, PEARL HARBOR DRY DOCK.



Pumping Crib I.—The first section of coffer-dam was completed during April, 1911, and enclosed 180 lin. ft. of the inboard end of the dock. Unwatering was begun on April 29th, 1911; the level of the water inside the coffer-dam was held from 2 to 3 ft. below sea level until May 3d while leaks in the sheet-piling were being located and stopped by divers. During the next 2 days the water was lowered to 16 ft. below sea level, when a slight rising was noted in the crib. On May 5th the water surface was lowered to 17.5 ft. where it was held until May 8th, when it was lowered to 21 ft. The cribwork gave evidence of serious distress when the depth of 21 ft. was reached, and the pressure was relieved by quickly reducing the head to 16 ft. The depth of 16 ft. was maintained until May 10th, when it was increased to 17 ft.; within a few hours after reaching this depth there was a sudden increase in the rate of leakage into the coffer-dam, until it exceeded the combined normal capacity of 18 500 gal. per min. of the three 15-in. and one 8-in. drainage pumps, and caused the suspension of pumping operations. When pumping was discontinued, the water level within the coffer-dam rapidly arose nearly to tide level, the water apparently entering principally through an opening which had developed in the sheet-piling. Pumping was resumed on May 11th, but, after 2 days, using all four pumps, it was found impossible to unwater below 16 ft. due to the excessive leakage into the coffer-dam, and further effort to unwater the section was therefore abandoned.

Very peculiar and alarming movements in the sheet-piling and cribbing had been noted during pumping operations; on May 5th, under a head of 17.5 ft., the cribbing in the southwest quarter of the coffer-dam had risen about 4 in. and the sheet-piling in the south half of the west side had risen 3½ in.; on May 8th, with a head of 17.5 ft., the portions of crib and sheet-piling which had previously shown greatest movement had risen an additional ¾ in., and some rising had been noted over all other parts of the coffer-dam except in the northeast corner; after the head had been increased to 21 ft. on May 8th portions of crib and sheet-piling had come up 5½ in. farther and were rising so rapidly, having reached a maximum of from 9½ to 11⅛ in., according to different observers, that the coffer-dam would have failed had the head not been quickly reduced; there was an immediate settlement of 1 in. in the sheet-piles and crib when the head was reduced to 16 ft.; the crib settled 6½ in. and the sheet-piling 2½ in. when the coffer-dam

was flooded on May 10th; the maximum permanent distortion noted on May 18th, after the coffer-dam had been flooded for 5 days following the suspension of pumping operations, was a rise of 6 in. in part of the sheet-piling and a settlement of ¾ in. in the crib. Observations on the test column (base area, 4 sq. ft., loaded with 16 tons, and placed within the area of the coffer-dam to determine the bearing value of the soil) showed that the column rose slightly as the water in the coffer-dam was lowered, and settled when it was flooded.

Piling and Seal.—Experience acquired during the first attempt to unwater a section of the coffer-dam resulted in contract modifications, dated August 5th, 1911, which provided a piling foundation for the dock and permission to place concrete by the under-water method. The reason for the latter provision, and the limitation specified for its use, are given in the following quotation from the contract:

"For the purpose of forming a seal that will facilitate the unwatering of the excavation, concrete may be deposited under water. For this purpose tremies shall be used * * *. The depth of concrete so deposited shall not exceed 8 feet."

It was anticipated that the piles, which were to be 4 ft. from center to center longitudinally and about 3.5 ft. from center to center transversely, would have an average length of 39 ft., of which 4 ft. would be embedded in the concrete bottom of the dock, leaving 35 ft. for the depth of penetration. The piles would thus serve both as anchors and supports. The anchorage value of the piling naturally combined with the weight of the concrete seal to withstand the upward hydrostatic pressure on the concrete when the coffer-dam was pumped out; hence the greater the pile anchorage the less concrete required by the under-water method, and, as concrete placed through tremies was much more expensive than when deposited in the open, the less the cost to the contractors.

Contract payments on account of piling were based on the number of linear feet actually driven, and it was provided that the thickness of concrete in the bottom of the dock might be reduced 1 ft. to allow for the swelling of the ground which the driving of piles might produce. The reduction in the thickness of the concrete, as affecting the strength of the dock, was justified, inasmuch as the anchorage of the piling against hydrostatic uplift was more than equivalent to the weight of the concrete eliminated. The original plans for the dock

required that a 12-in. layer of rich concrete, 1 part cement to 4 parts stone, with enough sand to fill its voids, should form the outermost portion of the bottom and side-walls to serve as an impervious skin to exclude the percolation of ground-water into the interior mass of leaner 1:6½ concrete. The adoption of the under-water method for depositing concrete rendered it impossible to place these two grades as at first intended, and an average mix of about 1:3.4:6.19 was substituted, to be used alike for both tremie and dry concrete.

It was actually found that piles could not be driven to the depths anticipated, and the records show that the average penetration of the 1 966 piles driven in Section I, which includes 180 lin. ft. of dock at the inboard end, was 23.78 ft.; the 2 537 piles driven in Section II, which included 240 lin. ft. of dock adjoining Section I, have an average penetration of 20.53 ft.; the 2 652 piles driven in Section III have an average penetration of 28.49 ft.; and the 1 720 piles driven in Section IV have an average penetration of 28.20 ft. Pile driving was completed in Section I on September 11th, 1911; in Section II on December 2d, 1911; in Section III on June 13th, 1912; and in Section IV on January 11th, 1913. Piles are not yet driven for Section V. It was observed with surprise that, notwithstanding the close spacing and large number of piles driven, the surface of the ground between the piles showed little evidence of swelling or rising.

Placing concrete for the bottom of the dock and to form a seal in the bottom of the coffer-dam surrounding Section I was begun in October, 1911. It was soon found that the 1:3.4:6.19 mix was too lean to make satisfactory tremie concrete, and the use of a 1:3:5 mix was authorized on October 20th, 1911. Concreting through tremies in Section I was completed on January 4th, 1912; the 6 284 cu. yd. of concrete which were placed had an average thickness of 6.52 ft.

Pumping Crib I, Second Attempt.—The second attempt to unwater Section I of the dock was started on January 18th, 1912. The water surface was gradually lowered by four pumps having a nominal capacity of 18 500 gal. per min. A depth of 34.3 ft. was reached on January 26th, when it was noted that a portion of the cribbing of the coffer-dam near the west wall had risen 1 in. No further rise was noted in the coffer-dam 8½ hours later, when the head had been increased to 36.3 ft. The concrete in the bottom along the side-walls of the coffer-dam was exposed at this depth and found to be of generally poor quality.

and very poor in places. On January 28th the crib and sheet-piling near the center and west side of the coffer-dam had risen another inch, making a total of 2 in.; on January 28th, after the rise in the crib had increased to 2½ in., the head was reduced to 26 ft., where it was maintained until February 1st, while 160 tons of rock ballast were placed over the bottom at the places where the rise in the crib had been greatest. The head was then increased to 36.0 ft. without any further rise of the coffer-dam being observed. On February 2d the head was increased to 37.3 ft., and the removal of poor concrete and the substitution of richer concrete was started. Several leaks developed through the concrete seal on February 3d, and the pumps were stopped. The coffer-dam was completely flooded by leakage within 16 hours after pumping was stopped. On February 5th the crib had settled to within 1 in. of its original position on January 18th. On February 11th pumping was again started, and a head of 37.5 ft. had been obtained at 11.30 A. M. on February 13th when large leaks developed through the concrete, which resulted in pumping operations being stopped at 3.10 P. M., and the coffer-dam being allowed to fill. The defects in the concrete were apparently due to lack of experience in using the tremies under the conditions which prevailed and to the use of too lean a mixture, causing the impregnation of the concrete mass with sea water.

Concrete Changes.—The troubles experienced during the second attempt to unwater Section I led to contract changes dated January 2d, 1913, which required the removal of all the defective concrete previously placed in Section I, and provided for the use of a very much richer mixture for tremie-placed concrete. A considerable quantity of silicious sand from the California Coast was necessary for this new mixture. The increase in the dimensions of the dock covered by this contract change was made possible by the increase in the limit of cost which had shortly before been authorized by Congress.

The concrete seal made with the new and richer tremie mixture was completed in the coffer-dam surrounding Section II of the dock on January 20th, 1913. This section comprised the 240 lin. ft. of dock adjoining Section I, around which the coffer-dam had been constructed while concreting and pumping operations had been going on in Section I. The seal contained 8 555 cu. yd. of concrete having an average thickness of 6.75 ft. after making allowance for the volume

of embedded pile heads. In placing this seal, advantage was taken to provide a positive bond to connect the tremie concrete and the upper mass which was to be placed after unwatering; this bond was considered necessary to prevent the possible lifting of the upper mass of concrete in case leakage through the tremie concrete should produce internal disrupting pressures, and was to be accomplished by finishing the surfaces of the tremie concrete in adjacent forms at different levels; as the tremie concrete was poured into forms, each covering areas about 12 ft. square, this arrangement resulted in producing a surface somewhat like a checker board in which the surface of each 12-ft. square was approximately 12 in. above or below the surfaces of adjacent squares, thereby giving an effective dove-tailed bond with the superimposed concrete which was to be placed at a later date.

Pumping Crib II.—A final attempt to unwater the combination timber and concrete coffer-dam surrounding Section II of the dock was started on February 6th, 1913. Table 1 gives the depth of unwatering from day to day, as well as the corresponding total rise of crib noted at two points of maximum uplift.

TABLE 1.—Depth of Unwatering of Section II, With Corresponding Total Rise of Crib.

Date, 1913.	Hour.	Depth below.	Rise in Crib, in Feet:	
			At west wall.	At east wall.
Feb. 6	.....	0.00 ft.	.....	.....
" 11	3.30 P. M.	5.50 "	0.03	0.03
" 12	3.15 P. M.	9.50 "	0.04	0.04
" 13	8.15 A. M.	12.25 "	0.04	0.04
" 13	3.00 P. M.	16.50 "	0.05	0.06
" 14	8.00 A. M.	16.50 "	0.04	0.05
" 14	3.00 P. M	20.00 "	0.05	0.06
" 15	8.00 A. M.	24.50 "	0.05	0.06
" 15	1.00 P. M.	25.50 "	0.07	0.07
" 16	10.15 A. M.	32.50 "	0.08	0.07
" 16	3.00 P. M.	33.50 "	0.11	0.07
" 17	8.30 A. M.	34.00 "	0.18	0.10
" 17	9.45 A. M.	34.25 "	0.19	0.12
" 17	10.30 A. M.	34.75 "	0.19	0.10
" 17	1.00 P. M.	35.50 "	0.25	0.13
" 17	1.45 P. M.	37.05 "	.....	.....
" 17	2.30 P. M.	36.75 "	0.66	0.15
" 17	2.40 P. M.	36.50 "	.....	0.18
" 17	3.00 P. M.	Coffer-dam failed at about this time.		

The outer row of concrete blocks along each side-wall of the coffer-dam was exposed on February 16th, and the quality of the concrete was found to be excellent. About sixty 2½-in. vertical drain pipes had been placed in the concrete seal during its construction; these drains,

distributed over the area of the coffer-dam, all flowed to some extent when the coffer-dam was pumped out, and a few of them flowed freely; a piece of hose connected with the upper end of one of the free-flowing drains filled with water to within 1 ft. of low-tide level, indicating that the water pressure beneath the concrete seal was nearly equal to sea pressure at corresponding depth, and probably would have been fully equal to sea pressure had the other drainage pipes in the vicinity of the one under test been closed. It will be noted from Table 1 that the rate of uplift of the coffer-dam was not very great until the morning of February 17th, and did not begin to be very alarming until about 1.00 P. M., when it was evident that the work was in grave danger. One of the pumps was stopped at 1.45 P. M. and all pumping was stopped at 2.00 P. M. The coffer-dam collapsed at about 3.00 P. M.; the crib lifted several feet at the time of failure, according to the statements of eye-witnesses.

The portions of the coffer-dam surrounding Sections I and III of the dock failed with the failure of the crib in Section II. Fortunately, no lives were lost, nor was any one injured. Figs. 5 and 6 are photographs showing the work after the collapse. Comparison of soundings taken on the surface of the concrete seal before the collapse with those made 6 weeks thereafter, indicates local vertical displacements varying from a maximum uplift of 9.4 ft. to actual settlements. It has not been practicable to obtain any accurate knowledge regarding the movements in the piling which supported and anchored the seal, but it seemed safe to conclude that it had all been rendered unreliable and therefore practically worthless, and the recently approved plans require its entire renewal.

Pressures.—The weight of excavated material removed from the dock site probably varied from about 2 500 lb. per sq. ft. at the entrance end, where the natural surface was nearly 10 ft. below low-tide level, to about 4 500 lb. per sq. ft. at the inner end of the dock, where the original surface was 14 ft. above tide level. The side slopes of the excavation stood at a very steep angle after the completion of the excavation. These slopes did not seem to be materially disturbed as a result of the coffer-dam, pile-driving, and pumping operations, or by the collapse of the coffer-dam, and there is no evidence of any material subsidence of the side banks. The submerged weight of the concrete in the completed dry dock structure, if considered as uniformly dis-

FIG. 5.—WRECK OF COFFER-DAM AFTER COLLAPSE, PEARL HARBOR DRY DOCK.

FIG. 6.—WRECK OF COFFER-DAM AFTER COLLAPSE, PEARL HARBOR DRY DOCK.

tributed, will be about 2 000 lb. per sq. ft. of area; if the weight of the side-walls is considered separately from the weight of the floor, the unit weight, per square foot of area covered, will be about 3 300 lb. under the side-walls and about 1 400 lb. under the floor. With the dock pumped out and containing a ship of maximum dimensions, the uniformly distributed weight on the underlying soil, in excess of full hydrostatic uplift corresponding to low-tide level, will be about 1 500 lb. per sq. ft.; if the weight of the side-walls is considered separately from that of the floor and ship, the weights imposed under the side-walls and floor will be about 2 300 and 1 100 lb. per sq. ft., respectively.

It is of interest to compare the maximum depths of 37.5 and 37.05 ft. obtained on February 11th, 1912, and February 17th, 1913, in the coffer-dams of Section I and Section II, respectively, with the depth of 38 ft. between extreme high-tide level and the floor of the completed dock. Although the concrete which had been placed in Section I was only 6.52 ft. thick, and in Section II only 6.75 ft. thick, as compared with the thickness of 16 ft. required for the finished dock, and although the side-walls which would tend to anchor the floor and reduce its span had not been started, it will be seen that the partly completed floor in both cases had been called on to resist nearly the maximum uplift which could be brought to bear on the completed structure.

Full hydrostatic uplift on the bottom of the dock at the time of collapse would average about 3 100 lb. per sq. ft. of area. Forces resisting this uplift were developed by the weight of concrete in the seal, the anchorage of the piles, the weight of coffer-dam materials, the weight of water remaining in the coffer-dam, the arch action developed in the concrete seal, and the arch action developed in the layer of soil which was penetrated by the piles; these resisting forces averaged, per square foot, about 1 012 lb. on account of concrete, about 1 080 lb. on account of piles (using 54 lb. per cu. ft. as a conservative estimate for the submerged weight of soil), about 170 lb. on account of coffer-dam, and about 300 lb. on account of water remaining in the coffer-dam; no accurate estimate can be made of the arch resistance in either the concrete or soil; from this analysis it would appear that arch resistance to uplift amounted to about 538 lb. per sq. ft. at the time of collapse.

Although the foregoing analysis based on averages is very interesting, it is extremely doubtful whether the figures given represent the facts due to the following conditions: the dish-shaped form of the concrete seal, which made the unit hydrostatic uplift along the sides much less than under the middle portions; the great differences in the thickness of the concrete blocks which composed the seal and in the length of the anchoring piles; the difference in the depth of water remaining in the coffer-dam at different points, which varied from about nothing at the sides to a depth of several feet over the middle portions; and to the irregular character of the soil underlying the coffer-dam. It is very probable that the collapse of the coffer-dam originated at some particularly weak point in the seal or in the underlying soil, and that local weakness resulted in general failure.

Concrete.

Materials.—The Island of Oahu, unfortunately, does not abound in materials which are best suited for making concrete. Gravel is not obtainable, and there is no silicious sand. A very satisfactory broken stone is made by crushing a hard blue lava rock, but it has been found very difficult to locate a quarry having the necessary capacity and transportation connections, and at the same time being comparatively free from the unsuitable grade of porous rock which is known locally as "puka-puka". There is an abundance of coral and sea-shell beach sand, locally called "Waianae", but its use for concrete is objectionable because of its uncertain chemical stability when incorporated in the completed dock. Screenings from the stone crusher are used to a large extent in lieu of sand. Cement is obtained from California mills.

Sand.—Fig. 7 indicates graphically the character of the natural sands and of the artificial substitute products, obtained either from the stone crusher or by mixing, which were considered as available for use in making concrete. Curve 1 represents a silicious building sand of medium fineness obtained from Puget Sound. Curve 2 represents the screenings obtained through a ¼-in. sieve from the rock crusher which supplies broken stone, and includes crusher dust. Curve 3 represents a combination of the materials used for Curves 1 and 2 mixed in proportion of one part of the former with two parts of the latter. Curve 4 represents crusher screenings through a ¼-in. sieve and is the

same as shown by Curve 2, except that dust passing a 30-mesh sieve is removed. Curve 5 represents coral sand obtained from the vicinity of the work. The mixture represented by Curve 3 was adopted as the best, and was used in making the successful tremie concrete. A combination of equal parts of the materials represented by Curves 4 and 5 shows practically the same characteristics as the mixture represented

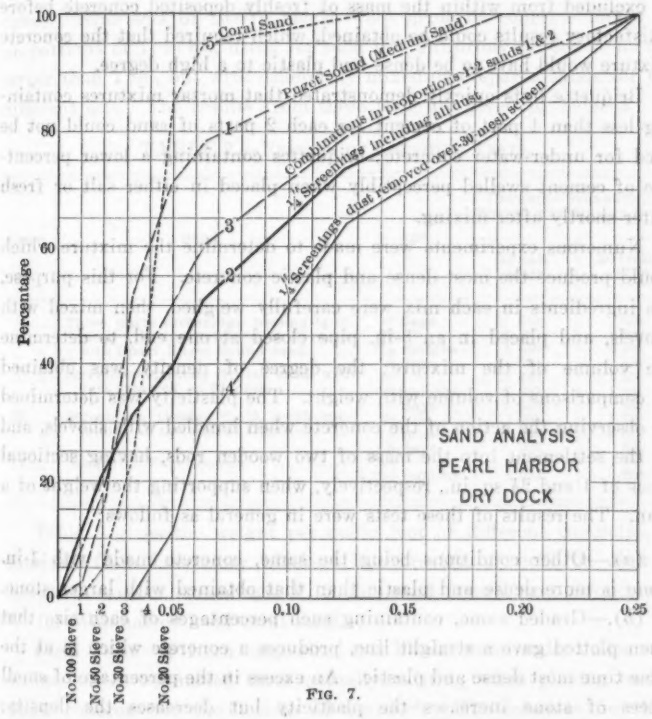


FIG. 7.

by Curve 3 for grains retained on a 20-mesh sieve, but the former contains only 7% of smaller grains which pass this sieve, as compared with 38% of the smaller-sized grains in the latter mixture; the larger percentage of small grains is necessary to obtain concrete of the best quality.

Mixture.—Examination of the concrete which was first placed by the under-water method revealed such serious defects as to render it

practically worthless. There was apparently a lack of bond throughout the mass due to general permeation with sea water, forming a soft, white, chalky substance with the cement which encased the particles of sand and pieces of broken stone in such way as to prevent consolidation. Chemical analysis showed that this chalky substance consisted principally of sulphates. It was evident that salt water would have to be excluded from within the mass of freshly deposited concrete before satisfactory results could be obtained, which required that the concrete mixture would have to be dense and plastic to a high degree.

Briquette tests quickly demonstrated that mortar mixtures containing less than 1 part of cement for each 2 parts of sand could not be used for under-water concrete; briquettes containing a lower percentage of cement swelled perceptibly when placed in either salt or fresh water shortly after mixing.

Numerous experiments were made to determine the mixture which would produce the most dense and plastic concrete. For this purpose, the ingredients in each mix were carefully weighed, then mixed with shovels, and placed in an 8-in. pipe closed at one end, to determine the volume of the mixture; the degree of density was obtained by comparisons of volume with weight. The plasticity was determined by observing the action of the concrete when handled with shovels, and by the settlement into the mass of two wooden rods, having sectional areas of 1 and 2½ sq. in., respectively, when supporting the weight of a man. The results of these tests were in general as follows:

(*a*).—Other conditions being the same, concrete made with 1-in. stone is more dense and plastic than that obtained with larger stone.

(*b*).—Graded stone, containing such percentages of each size that when plotted gave a straight line, produces a concrete which is at the same time most dense and plastic. An excess in the percentage of small pieces of stone increases the plasticity but decreases the density, whereas a deficiency in the small sizes has little effect on the density but reduces the plasticity.

(*c*).—Density and plasticity are a maximum when using Puget Sound sand mixed with screenings which contain crusher dust; elimination of any part of the crusher dust affects the results adversely. One part of Puget Sound sand with from two to three parts of screenings with dust gives the maximum plasticity, but the density is not

affected materially when the proportions are varied between the limits of 1:1 and 1:3.

(*d*).—Mixtures in proportions of 1:2.5:4 are more plastic than those of 1:2:4, but are not as dense; 1:2:4 mixtures are more dense than those of 1:2:3½, but are not as plastic.

Laboratory Tests.—A series of 132 crushing tests were made on 6-in. cubes. Nine sets of twelve blocks were made of concrete mixed in proportions of 1:2:3.5, using for each test uniformly graded stone not larger than 1 in., but with differently mixed or prepared sands for the different sets. The results from these tests are given in Table 2.

TABLE 2.—RESULTS OF CRUSHING TESTS.

Class.	Sand.	Age.	Crushing Strength, in Pounds per Square Inch:	
			In salt water.	In fresh water.
I....	Three parts screenings passing ½ in. mesh with dust retained on 30-mesh, with 1 part Puget Sound sand.....	3 days	685	620
		28 "	1 435	1 320
		2¼ months	1 850	2 285
II....	Screenings passing 3/16-in. mesh with dust retained on 30-mesh.....	3 days	590	470
		28 "	1 280	1 230
		3 months	1 985	1 985
III....	Screenings passing 3/16-in. mesh containing all the dust of fracture.....	3 days	630	565
		28 "	1 505	1 390
		3 months	2 655	2 370
IV....	Sand from Puget Sound.....	3 days	905	845
		28 "	1 745	1 780
		2¼ months	2 450	2 360

Table 3 gives the weight per cubic foot of different concretes, as obtained from laboratory blocks. For brevity, cr. will be used to designate crusher screenings, P. S. for sand from Puget Sound, and W. for Waianae or local coral sand.

Field Tests.—Fifteen large-sized test blocks were made, under conditions which resembled those actually existing when placing underwater concrete for the bottom of the dock. The concrete for these tests was deposited with tremies in water about 52 ft. deep. The blocks were about 4.5 ft. wide and 6.5 ft. long. The end of the tremie was held near one corner of the form so that concrete had to flow to an extreme distance of about 6 ft. after leaving the tremie. A 2-cu. yd. Ransome mixer was used for the concrete, which was made very wet, but with water limited so that it would not rise to the surface during transportation from mixer to tremie. The results from

TABLE 3.—WEIGHTS OF CONCRETE PER CUBIC FOOT.

Block No.	Mix.	Sand.	Stone.	Condition.	Weight
501.....	1 : 3.6 : 6.5	2 cr. ½ in. to dust 1 P. S.	2½ in. to ¼ in., graded..	24 hours, moist air.....	148.75
501.....	"	"	"	16½ hours, water.....	151.25
501.....	"	"	"	2 days, air.....	148.125
501.....	"	"	"	3 " "	147.5
502.....	"	"	"	24 hours, moist air.....	151.25
502.....	"	"	"	16½ hours, water.....	151.25
502.....	"	"	"	2 days, air.....	152.5
502.....	"	"	"	3 " "	152.5
9-D..	"	cr. ½ in. to dust.....	2½ in. to ¼ in., ungraded	Wet.....	137.5
10-D..	"	3 cr. ½ in. to dust 1 W..	"	"	146.5
11-D..	"	2 cr. ½ in. to dust 1 W..	"	"	150.0
12-D..	"	3 cr. ½ in. to dust 2 W..	"	"	147.6
21-D..	"	cr. ¼ in. to dust.....	"	"	147.7
22-D..	"	3 cr. ¼ in. to dust 1 W..	"	"	149.7
23-D..	"	2 cr. ¼ in. to dust 1 W..	"	"	150.5
24-D..	"	3 cr. ¼ in. to dust 2 W..	"	"	150.0
33-D..	"	cr. 5/16 in. to dust.....	"	"	157.0
34-D..	"	3 cr. 5/16 in. to dust 1 W..	"	"	158.2
35-D..	"	2 cr. 5/16 in. to dust 1 W..	"	"	157.0
36-D..	"	3 cr. ¼ in. to dust 2 W...	"	"	149.8
36-1D.	"	3 cr. 5/16 in. to dust 2 W..	"	"	153.0
42-D..	"	3 cr. ½ in. to dust 1 W..	2½ in. to ¼ in., graded..	"	155.0
45-D..	"	cr. ½ in. to dust.....	"	"	153.0
46-D..	"	3 cr. ½ in. to dust 1 W..	"	"	160.0
47-D..	"	2 cr. ½ in. to dust 1 W..	"	"	160.5
48-D..	"	3 cr. ½ in. to dust 2 W..	"	"	161.0
56-D..	"	3 cr. to dust 2 W.....	"	"	150.3
57-D..	"	cr. ¼ in. to dust.....	"	"	151.0
58-D..	"	3 cr. ¼ in. to dust 1 W..	"	"	154.0
59-D..	"	2 cr. ¼ in. to dust 1 W..	"	"	155.0
67-D..	"	2 cr. 5/16 in. to dust 1 W.	"	"	154.8
69-D..	"	cr. 5/16 in. to dust.....	"	"	148.0
70-D..	"	3 cr. 5/16 in. to dust 1 W..	"	"	154.0
72-D..	"	3 cr. 5/16 in. to dust 2 W..	"	"	150.3
79-D..	"	cr. ½ in. to dust.....	"	"	145.2
80-D..	"	cr. ¼ in. to dust.....	"	"	148.7
81-D..	"	cr. 5/16 in. to dust.....	"	"	153.0
503.....	1 : 3 : 6.5	2 cr. ¼ in. to dust 1 P. S.	"	24 hours, moist air.....	153.125
503.....	"	"	"	16½ hours in water.....	153.75
503.....	"	"	"	2 days in air...	150.62
503.....	"	"	"	3 days in air...	153.75
504.....	"	3 cr. ¼ in. to dust 2 P. S.	"	24 hours, moist air.....	152.5
504.....	"	"	"	16½ hours, water.....	151.25
504.....	"	"	"	2 days in air...	151.87
504.....	"	"	"	3 days in air...	151.87
1-D..	"	cr. ½ in. to dust.....	2½ in. to ¼ in., ungraded	Wet.....	136.1
3-D..	"	2 cr. ½ in. to dust 1 W..	"	"	146.3
4-D..	"	3 cr. ½ in. to dust.....	"	"	145.3
13-D..	"	cr. ¼ in. to dust.....	"	"	140.0
14-D..	"	3 cr. ¼ in. to dust 1 W...	"	"	149.0
15-D..	"	2 cr. ¼ in. to dust 1 W...	"	"	151.5
25-D..	"	cr. 5/16 in. to dust.....	"	"	149.5
26-D..	"	3 cr. 5/16 in. to dust 1 W..	"	"	154.0
27-D..	"	2 cr. 5/16 in. to dust 1 W..	"	"	153.1
32-D..	"	3 cr. 5/16 in. to dust.....	"	"	155.0
37-D..	"	cr. ½ in. to dust.....	2½ in. to ¼ in., graded..	"	152.0
38-D..	"	3 cr. ½ in. to dust.....	"	"	154.0
39-D..	"	2 cr. ½ in. to dust 1 W..	"	"	154.5
40-D..	"	3 cr. ½ in. to dust 2 W..	"	"	154.0
49-D..	"	cr. ¼ in. to dust.....	"	"	143.0
50-D..	"	3 cr. ¼ in. to dust 1 W..	"	"	150.0

TABLE 3.—(*Continued.*)

Block No.	Mix.	Sand.	Stone.	Condition.	Weight.
51-D..	1:3:6.5	2 cr. ¼ in. to dust 1 W..	2½ in. to ¼ in graded..	Wet	146.5
61-D..	"	3 cr. 3/16 in. to dust 1 W..	"	"	142.7
62-D..	"	3 cr. 3/16 in. to dust 1 W..	"	"	149.5
63-D..	"	2 cr. 3/16 in. to dust 1 W..	"	"	149.0
64-D..	"	3 cr. 3/16 in. to dust 2 W..	"	"	148.0
73-D..	"	cr. ½ in. to dust.....	"	"	140.0
74-D..	"	cr. ¼ in. to dust.....	"	"	149.0
75-D..	"	cr. 3/16 in. to dust.....	"	"	151.0
547.....	1:2:3.5	2 cr. ¼ in. to dust 1 P. S.	.....	"	151.25
547.....	"	"	.....	24 hours in air.	150.625
547.....	"	"	.....	16½ hours, water.....	152.5
547.....	"	"	.....	2 days in air...	152.5
547.....	"	"	.....	3 days in air...	152.5
505-D..	"	"	.....	24 hours moist air.....	153.7
505-D..	"	"	.....	16½ hours, water.....	152.5
505-D..	"	"	.....	2 days in air...	151.87
505-D..	"	"	.....	3 days in air...	153.75
506-D..	"	"	.....	Wet	151.0
507-D..	"	1 3/16 washed, cr. 1 P. S..	.....	"	155.0
508-D..	"	2 pt. fine, cr. 1 P. S.....	.....	"	157.0
509-D..	"	2 cr. 3/16 in. to dust 1 W..	.....	"	149.3
510-D..	"	cr. 3/16 in. to dust 1 P. S..	.....	"	153.2
511-D..	"	P S.....	.....	"	151.5
512-D..	"	cr. 3/16 in. to dust.....	.....	"	148.0

eight of these large test blocks, and also from small cubes made from the same mixes which were seasoned in the laboratory, are given in Table 4.

The concrete in Block No. 2 was fairly smooth-flowing and plastic. The surface of the block when raised for examination was rough, but hard and compact.

The concrete in Block No. 9 was free-flowing, sticky, and generally excellent in appearance. When the block was removed from the water it was found that it had perfectly filled the form, and that the grain of the wood in the form was distinctly reproduced in the sides of the block; the top of the block was flat and nearly level. In short, this specimen was the equal of high-grade concrete placed under favorable conditions on shore. The concrete in this block weighed 151.2 lb. per cu. ft.

The concrete in Block No. 12 did not show any serious separation of materials, but it was not of uniform texture, and had not hardened satisfactorily after an immersion of 24 days. It weighed 147 lb. per cu. ft.

TABLE 4.—Results of Tests of Large Test Blocks.

No.	Diameter of tremie, in inches.	Mix.	Sand.	Stone.	Age, in days.	Crushing Strength, in Pounds per Square Inch.	
						Tremie block.	Laboratory block.
2	15	1 : 2 : 3½	⅜-in. screenings retaining dust	1-in. well graded	30 36 90	860	1 355 2 030
9	15	1 : 2 : 3½	2 parts ¼-in. screenings retaining dust. 1 part Puget Sound	1-in. well graded	30 90 120	2 075 ..... 2 113	1 960 2 655 .....
12	15	1 : 2 : 3½	1 part ⅜-in. screenings and dust passing 30-mesh removed. 1 part Puget Sound	1-in. well graded	24	1 205	None
10	15	1 : 2 : 3½	1 part ⅜-in. screenings containing dust. 1 part Puget Sound	1-in. well graded	105	2 485	None
11	15	1 : 2 : 3½	Puget Sound	1-in. well graded	16 30 75	1 325 1 670 2 350	1 405 1 700 2 410
3	18	1 : 2 : 3½	2 parts ¼-in. screenings containing dust. 1 part coral.	1-in. well graded	28 38 68 90	..... 1 010 2 030	1 040 .... 1 570
8	18	1 : 3 : 5	2 parts ¼-in. screenings containing dust. 1 part coral.	1½-in. not graded	....	No tests	
13	15	1 : 2 : 4	Puget Sound	1½-in. not graded	....	No tests	

The concrete in Block No. 10 was very free-flowing and plastic, and presented a good appearance when raised for examination.

The concrete in Block No. 11 very much resembled that in Block No. 10, except that it was a little less plastic. It weighed 149.4 lb. per cu. ft.

The concrete in Block No. 3 was free-flowing and plastic, but it was found that there was considerable washed stone in the bottom of the form, which had accumulated during the early stage of the pouring, and that there were soft pockets in the block where the coral sand had segregated excessively.

The concrete in Block No. 8 was ashy in appearance, and was not plastic. The block was very inferior, and the ingredients were badly washed and separated.

The concrete in Block No. 13 was mixed moist, but not wet, with the result that the tremie choked repeatedly; the concrete did not flow smoothly to fill the form, and sea water entered, not only into the deposited mass, but also into the tremie; the result was failure, as far as obtaining a satisfactory concrete was concerned.

The mixture and method of placing the tremie concrete used in the coffer-dam surrounding Section II were the same as used for Block No. 9.

Tremies.—Approximately, 14 840 cu. yd. of concrete were deposited through tremies in water varying in depth from about 54 to 44 ft. A great deal of interesting experience in the use of tremies has been incidentally acquired, due to the fact that the product, which at first was practically worthless, was gradually improved until finally it attained a high degree of excellence. This improvement followed careful experiments with tremies of various diameters, manipulated in various ways, and handling concrete mixed in many different proportions with variously graded and selected materials. Tremies 12, 15, and 18 in. in diameter were used in these experiments, and the results seem to justify the following general conclusions:

(*a*).—The results from a tremie depend to a great extent on the proportions, character of materials, and plasticity of the concrete which is being used.

(*b*).—The excessive frictional resistance to the movement of concrete in a 12-in. pipe causes frequent clogging in the pipe and greatly reduces the pressure at the exit. Under these conditions, it becomes impracticable to hold the end of the tremie embedded in the deposited concrete, thereby bringing all newly placed concrete into direct contact with sea water, with consequent injury to the entire product.

(*c*).—The frictional resistance to the movement of concrete in an 18-in. pipe is not sufficient to prevent the occasional loss of a charge in the tremie, thereby interrupting the filling of a form, with the added uncertainty as to the quality of the product. There is no difficulty in maintaining the end of this tremie embedded in concrete, and readily filling the form without any disturbance of the mass which would admit sea water.

(*d*).—The frictional resistance in a 15-in. tremie is apparently just about right to obtain the proper discharge pressure necessary for efficiently regulating the flow of concrete by raising and lowering the tremie with the end maintained within the deposited mass. No difficulty is experienced, when using this tremie, due to short interruptions in the supply of concrete from the mixer. Concrete discharged from this

tremie flows freely to distant parts of the form, without causing disturbance in the mass which would admit sea water, and without any tendency for sea water to rise in the tremie.

(*e*).—A tremie 15 in. in diameter is best suited for the work, and was adopted and actually used for placing the greater part of the tremie concrete.

The 15-in. tremie consisted of a riveted steel pipe, with a funnel-shaped steel hopper, about 5 ft. in diameter, bolted to the upper end. The tremie hopper was suspended from a timber storage hopper having a capacity of about 2 cu. yd., and fitted with a 12-in. valve in the bottom for controlling the flow of concrete into the tremie. Concrete was mixed in a 2-cu. yd. mixer at the rate of from 40 to 50 batches per hour, each batch producing about 1.85 cu. yd. Concrete from the mixer was discharged into 2-cu. yd. bottom-dump tubs, which were transported on flat cars hauled by cable to a derrick, thence into the tremie storage hopper. When beginning operations, the tremie was set with its lower end resting on the bottom of the form and then raised from 2 to 3 in. A bag loosely filled with straw was then placed in the tremie hopper to serve as a piston which descended under the burden of concrete to force the sea water out of the tremie. It was customary to assemble about 6 cu. yd. of concrete at the derrick before starting to fill the tremie, as it was found that this quantity when rapidly poured was sufficient to charge the tremie fully and embed its end to a depth of from 1.5 to 3.0 ft., and effectually seal the tremie from the entrance of sea water.

Special effort was made, after the tremie was first properly charged, to supply the concrete required to fill the form quickly and regularly; by carefully manipulating the depth of the tremie to correspond with its supply of concrete, it was found that the surface of the concrete in the form could be maintained in an almost flat and level condition, with uniform displacement throughout the form, and with the exposure throughout the process of only one unvarying surface to the injurious effect of contact with sea water. The forms for tremie concrete were each about 12 ft. square, and were of rough 1-in. boards and 2 by 4-in. studding; after framing the pieces for one side of a form on the bank, it was sunk in a vertical position to its proper place; the sides of the form were then closely joined together and braced by a diver, who closed any openings in the bottom with concrete in bags and also made

sure that the form was properly cleared and ready for concrete just before pouring was begun. Approximately 40 cu. yd. of concrete were required to fill one form.

General Tests and Investigations.

Borings.—Numerous borings have been made at the dock site to determine sub-surface conditions. These consist of a series of 38 borings of the wash type which were made in 1908, before work on the dock was started, and a series of 18 of the core type made in 1913.

The preliminary borings were on the axial line of the dock, and on two parallel lines about 65 to 75 ft. on each side of the axis. The borings were at intervals of about 100 ft. on these lines, and were arranged so that there was one on each side line opposite one on the center line, to facilitate the making of cross-sections. The original ground level over the dock site varied between an elevation of 14 ft. above mean low-tide level to 8 ft. below. The preliminary borings were carried to about the same average depth of 75 ft. below tide level, or about 28 ft. below the bottom of the dock. The borings were made by driving a casing pipe into the ground, loosening the penetrated material by blasting and churning when necessary, and then removing the contents of the casing with a water jet from a small pipe within. The only samples of the underlying soil which could be obtained by this method were the deposits found in the bottom of a bucket in which the overflow from the casing was collected; due to the disintegration of the material and the removal of the lighter particles which this process involved, there was necessarily a considerable difference between the samples and the original formations which they represented.

There were several special reasons for undertaking the second series of borings made in 1913:

(*a*) To check the results found by the 1908 series.

(*b*) To find, if possible, an explanation for the peculiar movements of the coffer-dam on the three occasions when efforts were made to unwater the excavation.

(*c*) To determine more definitely the suitability of the site for the construction of a graving dock.

(*d*) To determine the weight of the materials in the various strata penetrated.

The new borings, therefore, were located near the points which had been selected for the preliminary borings, were carried to considerably greater depths than those reached by the earlier series, and were made by methods which permitted of obtaining much more accurate information regarding the materials penetrated.

The first fourteen borings of the 1913 series were made by using a 6-in. casing pipe in combination with the churning and jet devices which comprise the usual wash-boring outfit; a heavy driving rod fitted at the lower end with a short removable piece of 3-in. pipe or core barrel was utilized for obtaining samples. With this equipment it was found possible to drive the core barrel to undisturbed depths below the bottom of the casing, and obtain cores as frequently as desired, provided the material was of a nature or consistency which would remain in the core barrel while being raised. The core barrel was provided with holes above the highest point reached by the cores to prevent the forming of internal water pressure by an entering core which would tend to disintegrate or modify the character of the core when it was withdrawn. These fourteen borings ranged in depth from 84 to 192 ft., with an average of 144 ft. below low-tide level. It was found in practice that considerable force was frequently required to remove cores from the core barrel, and that probably the physical character, and especially the density, of the material was thereby affected.

The last four borings of the 1913 series were made with a rotary Calyx drilling outfit having dimensions for obtaining 2-in. cores. Cores obtained with this apparatus were not compressed or modified while being secured, and more truly represented the original formations than those obtained with the driving rod. These four borings reached depths varying from 94 to 107 ft., with an average depth of 99 ft. below low-tide level.

While certain borings were in progress, and when the top of the casing tube was only a short distance above sea level, it was noted that there was an overflow of fresh water from the casing; by increasing the elevation of the top of the casing it was found that the height to which the fresh water would rise could be computed by multiplying the depth of the water-bearing stratum by the ratio between the weights of salt and fresh water; in other words, the phenomenon was

not of Artesian origin, but indicated the presence of ground-water below an impervious stratum.

With due allowance for the very irregular character of the subsoil, the results from the 1913 series of borings were in close accord with those found from the preliminary borings in 1908. The mud stratum beneath the dock is apparently impervious and unbroken in its continuity, but varies in thickness from about 4 to 40 ft. The borings gave no evidence of unusual compressibility or of fluidity in any of the underlying formations.

The character of the mud stratum has been of vital importance throughout this work. The original design for the dock and the method selected by the contractor for its construction were undoubtedly based on the belief that the stratum was continuous, solid, impervious, and comparatively non-compressible. The stratum apparently has all these qualities in its original form, but it is probable that the conditions of solidity and imperviousness were impaired by building operations. In the writer's opinion, all the trouble which has been experienced since the work was started may be attributed to insufficient solidity of this stratum due to thin portions and to the breaking of the stratum by piling and by upheavals after removing the top cover. Figs. 8 and 9 are photographs of cores obtained from borings through the mud stratum; the plasticity of the material is indicated by the bands on the sample shown in Fig. 8, which formed when the core was driven out of the core barrel; Fig. 9 illustrates the tendency of the mud to check and crack when it dries. Samples of the mud when freshly obtained are plastic like putty or clay, but when immersed in water quickly and completely disintegrate like so much corn-meal. A sample placed on the tongue does not give any impression of grit. It is interesting to note that it is this material which forms the soil in which sugar cane thrives so wonderfully.

Bearing Tests.—Two tests were made to determine the bearing value of the soil in the bottom of the dry dock excavation. These tests were conducted before any piling had been driven for the support of the dock and after the excavation had been practically completed to the required depth at the points selected for the tests.

The first test, made in April, 1910, covered an area of 4 sq. ft. Under a load of 1.5 tons per sq. ft., there was a settlement of about 3 in. in 12 hours; the greater part of this settlement occurred immedi-

ately after the application of the load, as might have been expected with a load applied to soft ground. The load was then gradually increased, during a period of 25 min., until it amounted to 3 tons per sq. ft., under which the total settlement reached 5.25 in. after 29 hours, and was slowly increasing.

The second test was conducted in May, 1910, and was applied to a strongly trussed tripod having legs 10 ft. apart, each resting on a shoe having an area of 4 sq. ft. After 42 hours under a load of 1 ton per sq. ft., the settlements under each shoe were 2, $3\frac{3}{4}$, and $2\frac{1}{4}$ in., respectively. The load was gradually increased to 4 tons per sq. ft., which increased the settlements to $4\frac{1}{2}$, $9\frac{5}{8}$, and 5 in., respectively. After the load of 4 tons per sq. ft. had been supported for 24 hours, the settlements had increased to $4\frac{7}{8}$, $11\frac{1}{8}$, and $5\frac{3}{8}$ in.; after a further period of 17 hours under this load, the settlements had increased only $\frac{1}{100}$, $\frac{1}{100}$, and $\frac{1}{50}$ in., which are too small to be measured accurately with ordinary field instruments and for all practical purposes the tripod could be considered as having reached a condition of rest. The possible unequal distribution of the load to the three legs probably accounts in part for their unequal settlement, but the real cause of the inequality is undoubtedly due to a difference in the bearing value of the soil at the three points.

Pile Tests.—During May, 1911, five test piles were driven in the bottom of the coffer-dam surrounding Section I, and fifteen piles at various points in the bottom of the excavation outside of the coffer-dam. The penetration of the piles varied from 18.2 to 48.9 ft. After flooding the coffer-dam it was found that one of the test piles inside the coffer-dam had settled $\frac{3}{8}$ in., two had settled $\frac{1}{8}$ in., and one had remained without change; the record of the fifth pile has been lost. One of the piles inside the coffer-dam, with a penetration of 27.2 ft., was gradually loaded during a period of 3 hours with a load of 16 tons; the pile settled a total of $1\frac{5}{8}$ in. while being loaded, and, after carrying the load for 44 min., had gone down an additional $\frac{1}{8}$ in.; no more settlement had occurred 70 min. later.

Four tests were made during May, 1911, to determine the anchorage value or resistance to pulling of a single pile. The test piles had been driven in the bottom of the dock excavation, and were separated by considerable distances. It was found to be impracticable to apply

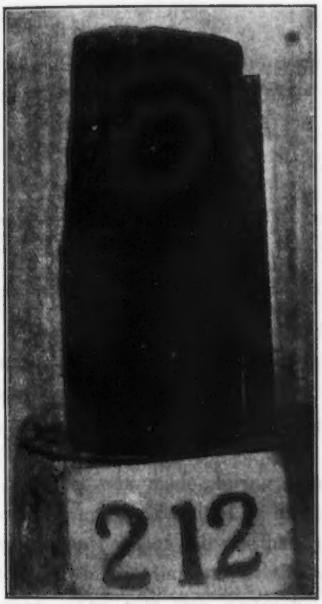


FIG. 8.—MUD CORE, SHOWING BANDS ON SURFACE WHEN DRIVEN OUT OF CORE BARREL.

FIG. 9.—MUD CORE, SHOWING CRACKS AND CHANGES IN APPEARANCE AS RESULT OF DRYING.

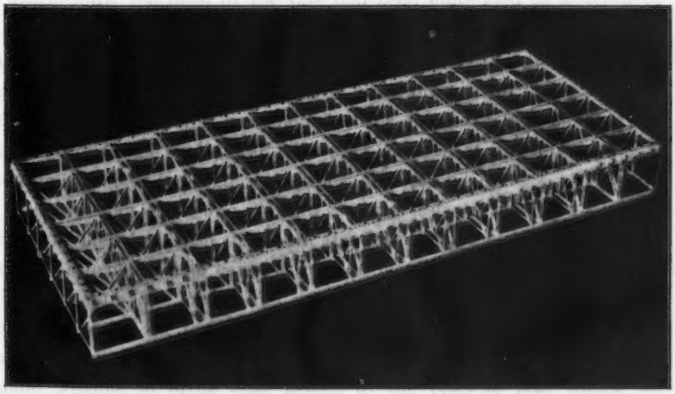
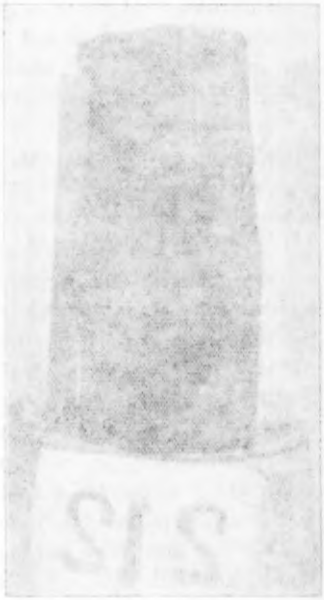
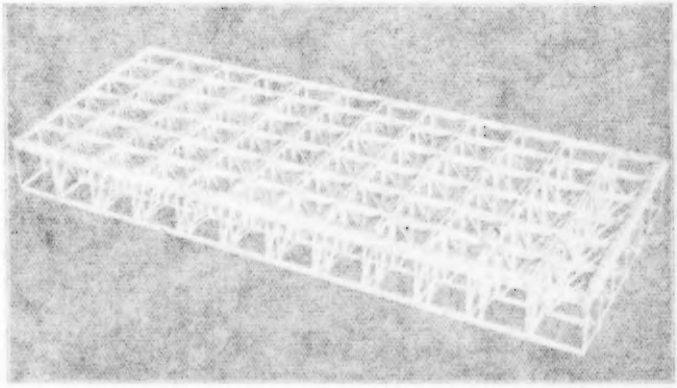


FIG. 10.—STEEL FRAME TO BE EMBEDDED IN BASE OF DOCK SECTION.





loads sufficient to draw the piles; the results of the tests are given in Table 5.

TABLE 5.—RESULTS OF TESTS IN PULLING PILES.

Pile No.	Penetration.	Pull.		Rise, in feet.	Permanent set.
		Tons.	Time, in hours.		
7	19.3 ft.	56	18	0.002	....
9	32.2 "	40	18	0.017	0.00
14	31.4 "	40	18	0.002	0.00
19	36.9 "	40	18	0.003	0.00

It is very probable that the "rise" of the piles under load was apparent rather than real, in view of the small movements noted and the complete recovery after removing the load, and were due either to observation errors or a slight straightening of the piles while under pull.

A test was made during February, 1913, to determine the pull which would be required to draw a pile from a block of concrete in which the head of the pile had been embedded. The pile was 45 ft. long, 12 in. in diameter at the butt, and 7.5 in. in diameter at the point. The pile was suspended in water with the butt pointing downward into a form resting on the bottom of the dock; concrete mixed in the proportions of 1:2:3.5 was poured into the form through a tremie until the head of the pile had been embedded to a depth of 5 ft. A direct and slowly increased pulling load was applied to the pile after the concrete had set for 31 days; when the pull amounted to 29.2 tons, the upper end of the pile broke off without any movement of the pile head having been noted; after getting a new hold on the pile the load was again gradually applied until it amounted to 47.0 tons, but without budging the pile head in the concrete.

During May and June, 1913, a loading test was made on a group of piles which had been driven in the bottom of the dock excavation. It had been planned to carry a grillage platform on four rows of five piles and apply the load on an upper deck above sea level, which was to be supported on the grillage by a braced trestle. Due to irregularities in spacing the piles in a depth of about 52 ft. of water, it was found possible to bring only fifteen of the piles to bear under the

grillage. The loading was begun on May 20th, and the load was gradually increased until May 31st, when it amounted to 400 tons; the load was allowed to remain until June 10th, and was then gradually removed during the next 4 days. The settlement slowly increased while supporting the fixed load, and amounted to 5.0, 1.1, 2.45, and 2.25 in., respectively, at the four corners of the platform at the time the unloading began. After the removal of the load there was a rise at the four corners of 3.25, 0.35, 1.25, and 1 in., respectively. After removing the trestle and grillage, it was found that the timber was badly crushed at various places, and there was evidence of deflection in the uprights; these conditions combined would probably account for a considerable part of the recorded settlements. This test applied an average load of about 26.7 tons to each of the fifteen piles, but, due to the crushing of the timber, there is much uncertainty as to the actual distribution of the load and the settlement of the piles under load. Observations on unloaded piles in the vicinity of the grillage showed settlements running up to 0.4 in., as the load was applied to the platform, but the latter was permanent after the removal of the load; these settlements would indicate a compression of the ground below the points of loaded piling.

During August and September, 1913, a second loading test was made on a group of twenty piles, in much the same manner as that followed during the preceding test, except that levels were taken on splines attached directly to the piles, and precautions were taken to eliminate errors which would result from the crushing or yielding of the temporary framework. Table 6 shows the results from this test. These results compare very closely with those obtained from the preceding test.

The specification for pile-driving provided for the use of a water jet, if directed by the officer in charge. In view of the impossibility of obtaining pile penetrations as great as were anticipated and as were desired for the adopted constructive methods, a special test was conducted during April, 1913, to determine the local practicability and value of the jet process. The test pile was 76 ft. long, 15 in. in diameter at the butt, and 9.5 in. in diameter at the point, and was driven approximately in the center of the dock excavation, in water 52 ft. deep. After 80 min. of jetting through a 2½-in. pipe, with a

TABLE 6.—Loading Tests of Piles.

Date, 1913.	Load period, in days.	Average load per pile, in tons.	Total settlement, in inches. Average of 20 piles.
August 29th	0	2	0.00
September 3d	5	8	0.42
" 4th	6	9	0.48
" 5th	7	11	0.55
" 6th	8	15	0.84
" 7th	9	15	0.97
" 8th	10	15	1.03
" 9th	11	17	1.20
" 10th	12	19	1.09
" 11th	13	20	1.16
" 12th	14	20	1.22
" 13th	15	20	1.25
" 14th	16	..	....
" 15th	17	22	1.25
" 16th	18	24.3	1.28
" 17th	19	24.3	1.61
" 18th	20	24.3	1.53
" 19th	21	24.3	1.59
" 20th	22	24.3	1.64
" 21st	23	24.3	1.61
" 22d	24	24.3	1.69
" 23d	25	24.3	1.75
" 24th	26	24.3	1.77

⅞-in. nozzle, under a pressure varying from 150 to 350 lb. per sq. in., the pile had sunk 2.1 ft. The pile was then struck 35 blows with a 5-ton, 42-in. stroke, steam hammer, which added 4.3 ft. to the penetration. With difficulty the pile was then drawn and the jet was inserted in the hole and churned for 45 min., with water pressure at 350 lb., with the result that the hole was filled to a depth of 4.4 ft. with loosened coral rock which the jet could not remove. The jet was then taken out and the pile re-inserted and alternately driven, pulled, and re-driven, until it reached a penetration of 12 ft. The pile was then again pulled and inserted with the jet pipe, with the nozzle at the point of the pile; a jet was applied at 350 lb. pressure, and the pile came to rest at a penetration of 5.5 ft.; after continuing the jet for 15 min., with the weight of the hammer resting on the pile, the penetration had increased ½ ft. An effort was then made to drive the pile while the jet was in operation, with the nozzle held at the pile point, but after eight blows the nozzle became clogged and the jet was removed. The alternate driving and pulling of the pile was then resumed, with gains of about 12 in. in penetration for each twelve blows, until a total penetration of 14 ft. had been obtained, when further operations were suspended because of the great difficulties attending the pulling process.

Caisson, Pumps, and Capstans.

Only brief mention will be made of the caisson, pumping equipment, and electric capstans which, although necessary accessories, do not form part of the graving dock structure proper and are furnished under separate contracts.

The caisson is of the floating boat type, and is equipped with electrically operated valves, pumps, and capstans. Electricity for the operation of the caisson will be obtained through portable connections with the station service. The caisson, built and delivered by the Union Iron Works, of San Francisco, has cost $133 923.93, and, properly modified to suit different tidal conditions, is almost identical in design with the caissons provided for Dock No. 4 at New York and for Dock No. 2 at Puget Sound.

The pumping equipment comprises four main discharge pumps and two drainage pumps. All pumps are of the vertical-shaft, volute type, each operated by a direct-connected, 3-phase, 2 200-volt, induction motor. The main pumps operate at 219 to 222 rev. per min., have a 54-in. suction, 48-in. discharge, and deliver approximately 8 000 cu. ft. per min. against 32 ft. head; 10 600 cu. ft. at 20 ft. head; 11 600 cu. ft. at 10 ft. head; and 11 800 cu. ft at zero head; the approximate input into the motor corresponding with this service is 700 h.p. at 32 ft. head; 586 h.p. at 20 ft. head; 475 h.p. at 10 ft. head; and 360 h.p. at zero head; considering that a motor and pump, together with its suction and discharge friction, combine to form a unit, the efficiency of a unit based on this performance is approximately 71.4% at 32 ft. head, 70.2% at 20 ft. head, and 47.4% at 10 ft. head. The drainage pumps operate at 585 rev. per min., having 20-in. suction and discharge, and, when operating against a head varying from 37.5 to 42.5 ft., deliver approximately from 675 to 560 cu. ft. per min., with an input varying from 74.7 to 71.7 h.p., or with a combined efficiency varying from 66.3 to 64.4 per cent. The pumps are below the floor of the dock, thereby eliminating all priming troubles. The motors are on a floor above tide level. The weight of the rotor, shaft, and impeller, of the main pumps, is carried by a collar bearing, just below the motor. The journals, bearings, and steady rests required for the pumping equipment are lubricated by a continuous system, through which oil is circulated under a pressure of about 180

lb. per sq. in. The pumping equipment for the Pearl Harbor dock is practically identical with that which has been installed in connection with Dry Dock No. 4 at New York and Dry Dock No. 2 at Puget Sound, and the foregoing operating data are based on results actually obtained during the operation of those docks.

The dock is to be equipped with nine capstans, one at the head of the dock and four along each side. These capstans will be used for hauling a vessel into the dock and for locating and centering the vessel correctly with reference to the keel blocks which have been prepared for its support. Each capstan is operated by a 50-h.p., 220-volt, induction motor, and the apparatus is designed to give a rope pull of 35 000 lb. at the rate of 12 ft. per min., or of 14 000 lb. at the rate of 30 ft. per min. These capstans are identical with those at Dry Dock No. 4 at the Navy Yard, New York.

Suggested Plans.

The long series of troubles and difficulties which had marked the progress of field work, and which culminated in the collapse of the coffer-dam on February 17th, 1913, naturally led to serious thought being given to other methods of construction and changes in plans which would tend to avoid building hazards and insure successful results.

Cox Plan.—L. M. Cox, M. Am. Soc. C. E., Civil Engineer, U. S. N., suggested two different schemes for the work. The first was presented in 1911, and contemplated the adoption of a sectional method of construction in which the units would consist of boats or structures which would be partly constructed on shore and then floated to position and sunk in a manner very similar to that independently suggested at later dates by Mr. Harris and Mr. Noble.

Mr. Cox's second scheme, submitted during the early part of 1914, urged the practicability of constructing the dock by the general methods and according to the general plans which were being followed at the time of the collapse of the coffer-dam by utilizing a ballast tank to assist in resisting hydrostatic uplift on the bottom of the unwatered coffer-dam. The dimensions of the proposed tank were to be as great as practicable, within the limits of the clear space between the side-walls of the completed dock and the end bulkheads of the

coffer-dam. The tank was to have a substantial interior framework, designed to enable it to carry its full capacity of sea water when resting on a grillage support on the floor of the coffer-dam and at the same time to serve as part of the coffer-dam bracing. The primary function of the tank was to provide a ready means of applying a temporary ballast or loading to hold down the tremie-placed concrete bottom of the unwatered coffer-dam during the period required to complete the concrete work in the dock floor and side-walls. Work under the tank would be conducted in alleyways under the structure; these alleyways would first be arranged through the supporting grillage in which concrete would be placed to the required height and on which the tank would in turn be supported while the grillage was removed and concrete was placed in the vacated spaces. After the completion of the concrete work within any coffer-dam, the latter would be flooded and the tank emptied and floated through a gateway in the coffer-dam bulkhead and placed in readiness for use in the next adjacent coffer-dam.

Gayler Plan.—Recommendations for changes were submitted immediately after the collapse by E. R. Gayler, M. Am. Soc. C. E., Civil Engineer, U. S. N. This plan proposed to construct the dock of large concrete blocks joined together with tremie concrete and with steel reinforcement. Each block would weigh about 150 tons, and would be made on the surface of the ground in the vicinity of the dock. The plan had its inception in the fact that a floating crane having a capacity for handling loads of 150 gross tons had been provided as part of the station's equipment, and could be made serviceable for placing the blocks. Any upheaval of the dock during construction or after completion was to be avoided by increasing the thickness of the floor, thus not only adding to the strength of the floor, but incidentally making the weight of the dock greater than any possible hydrostatic uplift. All foundation piling would be carefully cut off, and the bottom would be prepared to the correct form before beginning to place the blocks. Spaces about 2 ft. wide would be left between adjacent blocks, except where longitudinal reinforcing trusses were to be placed, where the spaces would be about 6 ft. wide; these spaces would be filled later with tremie-placed concrete. Anchorages for transverse reinforcement would be provided in specially prepared

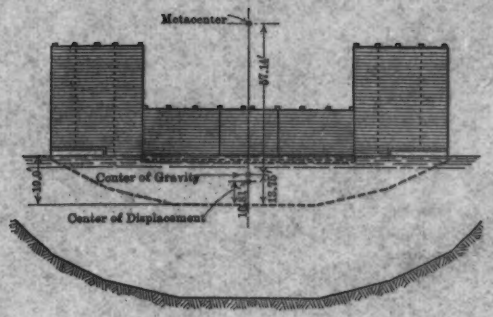


FIG. 1
CAISSON AFLOAT READY TO SINK

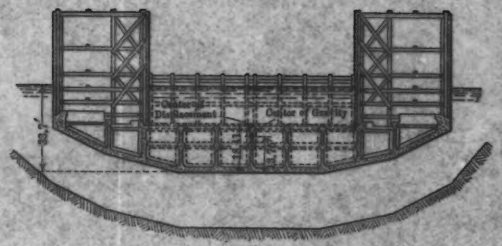


FIG. 2
CENTRAL PORTION FILLED WITH WATER TO SEA LEVEL

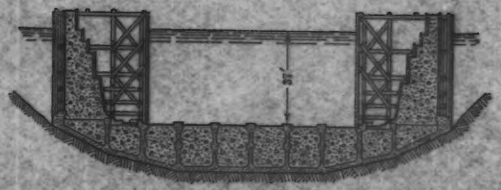


FIG. 3
SIDE-WALL COMPLETED

NOBLE DESIGN "TYPE A"

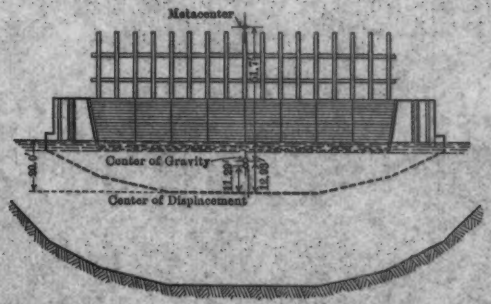


FIG. 1
CAISSON FLOATED, DRAFT 20 FT.
Fill 4½ ft. of water in 28 center pockets before floating, to reduce bending

FIG. 2
DRAFT 40 FT.
Build up side-wall pockets to next step, place 6½ ft. of concrete in pockets under walls and 5 ft. additional concrete in pockets of the two middle rows

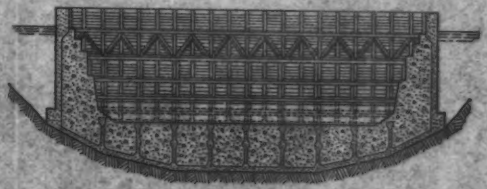


FIG. 3
ALL CONCRETE IN PLACE

NOBLE DESIGN "TYPE B"

GAYLER BLOCK TYPE

SKETCHES ILLUSTRATING METHODS OF CONSTRUCTION PROPOSED BY CIVIL ENGINEER GAYLER, U.S.N. AND MR. ALFRED NOBLE

anchor-blocks; after placing the anchor-blocks the connecting steel would be laid in the open spaces between the blocks and in grooves formed in the intermediate blocks; each transverse chain or tie would include a turnbuckle which would be tightened by a diver before embedding it in the concrete. Unequal settlements and cracks after the completion of the dock would be prevented by first allowing each block to settle independently to a position of rest before bonding it with its adjacent blocks, then combining the blocks in the side-walls, to permit the independent adjustment of the walls as a whole to positions of rest, then bonding the floor blocks with each other and with the side-walls, with the adjustment of the transverse tension bars. Plate VI shows the general dimensions and arrangements proposed by this plan.

Harris Plan.—F. R. Harris, M. Am. Soc. C. E., Civil Engineer, U. S. N., submitted two plans in a letter dated September 26th, 1913. One plan was based on continuing the work in accordance with the plans and method which were being followed at the time of the collapse of the coffer-dam, but proposed to obtain stability while the coffer-dam was unwatered by increasing the quantity and distribution of the tremie-placed concrete in the seal of the coffer-dam.

Mr. Harris' second plan proposed the adoption of a sectional-boat method, like that suggested by Mr. Cox in 1911. Mr. Harris recommended that the length of the dock should be separated into ten portions, approximately 99 ft. long, measured longitudinally with the dock. The base or hull for each unit was to be made of 1:2:4 reinforced concrete, and was to be constructed on launching ways. The bottom of the base was to be flat, thus differing from the dish-shaped, dry dock bottom required by the original contract. The general dimensions for the base were: over-all depth, 15.5 ft.; bottom, 8 in. thick; sides, 8 in. thick at bottom, increasing uniformly to about 14 in. at top; ends, 10 in. thick; wales at ends and sides, about 22 in. wide and 18 in. deep; solid bulkheads, 14 in. thick, about 9.7 ft. from center to center, transverse to the dock; solid bulkheads, 10 in. thick, about 21 ft. from center to center, longitudinal to the dock; an intermediate triangular buttress between each two longitudinal bulkheads to stiffen and support the sides; all corners filleted. The base, after launching, would be fitted with timber walls and bracing to form a coffer-dam having sufficient height to reach above

tide level when the base was sunk to rest on the prepared bottom of the dock excavation. On the completion of the coffer-dam, the boat would be towed to place and gradually sunk to its final position as concrete was added to fill the bottom pockets and form the side-walls.

It was proposed to use very lean concrete, 1:3.5:7 mix, for filling the pockets in the boat and 1:3:6 concrete for the side-walls; this involved dependence on the impermeability of the thin walls and the bottom of the boat to exclude seepage and prevent the possible chemical disintegration of the lean concrete in the dock bottom, but made no provision for the safeguard of the comparatively lean concrete in the side-walls. Any unequal settlement of the ten sections, which would render the finished dock unsightly, would be corrected by placing a 6-in. layer of concrete over the entire dock floor shortly before the completion of the work, and making the total thickness of the floor 16 ft. A comparatively small quantity of transverse steel reinforcement would be embedded near the upper surface of the dock floor, to care for certain structural stresses. The spaces between adjacent sections would be sealed in the bottom by a small quantity of tremie-placed concrete which would tightly wedge between the slightly upward converging faces of the bases; concrete placed through a tremie or by other special method would be used for making the vertical seals between the side-walls.

Great care would undoubtedly be required in the execution of this plan, to prevent launching stresses, which would cause cracks or breaks in the thin walls and bottom of the bases, and to protect the sections against injury while in a floating condition, particularly when heavily loaded just before settling into position. Great care would also be necessary to have the bottom of the dock excavation perfectly prepared before placing the sections and to prevent earth from the side banks or other foreign matter from being deposited on the prepared bottom before settling a base. Any error in locating a section or serious accident to a partly completed section would be extremely costly and difficult to repair, as it would be impossible to raise a section after it had firmly settled on the bottom.

Noble Plans.—During the summer of 1913 the Secretary of the Navy retained the late Alfred Noble, Past-President, Am. Soc. C. E., as Consulting Engineer on Pearl Harbor Dry Dock matters. Mr. Noble

visited the site of the dock during August to make such personal inspections and investigations as he might desire, to supplement the voluminous records and data which had been furnished him, and on October 20th, 1913, submitted a comprehensive detailed report with his recommendations. The report emphasized the desirability of eliminating as far as possible all features of design and of construction which were experimental or hazardous, or which involved uncertainties as to the result, and urged that cost be subordinated to safety.

Mr. Noble suggested two general methods for continuing the work, which he designated as the "Type A" and "Type B" methods, either of which he thought might be advantageously adopted, but expressed a decided preference for "Type A". Both methods propose sections each about 100 ft. long, in the direction of the axis of the dock, and founded on a reinforced concrete base, built above the surface of the water and then floated and sunk in the desired position. The use of a floating dock in which to construct the bases was strongly recommended in preference to launching ways, in order to eliminate as far as possible any injury to the structures during the process of floating them, and to permit of correcting any defects in construction which might appear after immersion but before the actual removal of the bases from the floating dock.

The five-plane, dish-shaped form required by the original contract for the contour of the bottom surface of the dock prism is retained in both plans proposed by Mr. Noble, but the thickness of the dock floor at the center is increased from 16 to 18.5 ft., to eliminate or reduce the need of reinforcing metal for tensile stresses. The increase in the thickness of the floor would correspondingly increase the required excavation; that is a matter of moment, considering the fact that the dredging work would have to be conducted at such great depth below the water surface and that closely spaced piles have been driven over about four-fifths of the dock area, which would greatly complicate the difficulties attending dredging operations. It would probably be necessary to remove interfering pile heads by blasting, inasmuch as the points of cut-off are so deeply buried in the ground; the points of cut-off would necessarily be several inches below the finished bottom grade level to permit of the accurate surfacing of the bottom to receive the concrete bases. The final accurate grading of the bottom to the rather complicated form required for the support of the bases would constitute

one of the principal difficulties in the execution of these plans. Only a narrow space, 2 or 3 in. wide, would be left between adjacent sections, and the sealing of these spaces would involve little difficulty because of their narrow dimension. The very strong and stiff construction of the bases would enable them to be adjusted to considerable irregularities in the supporting value of the ground, without endangering their integrity.

The boat or base proposed for the "Type A" method would have a concrete bottom 2 ft. thick, walls 1.5 ft. thick, and 12-in. solid interior longitudinal and transverse bulkheads, about 14 ft. from center to center. All concrete would be heavily reinforced. The upper part of each base would be designed to receive a timber coffer-dam at each end, within which the side-walls for the dock would be constructed; provision would also be made for temporary sheeting and cribbing, which would be placed on top of the base between the end coffer-dam for use during the transporting and sinking of the base to position. A distinctive feature of this plan, and one which Mr. Noble considered very important, was the requirement that any base after leaving the floating dock should be quickly transported and sunk to position, thus reducing to a minimum the period during which a base was afloat and safeguarding that part of the work against accidents which might be very serious in their results. The rapid sinking of a base would be obtained by the admission of water into the coffer-dam. The open pockets formed by the interior bulkheads in the base would be filled with tremie-placed concrete, and the side-walls would be made of concrete placed under open and dry conditions within coffer-dams.

The essential difference between the "Type B" plan and that described for "Type A" is the method proposed for sinking the bases. In "Type B" the coffer-dam for each section would cover the entire area of the boat, and the boat would be sunk by the weight of the concrete filling, all of which would be placed under dry and open conditions. This plan is very similar to that suggested by Mr. Harris. Mr. Noble considered that the long period during which a base would be maintained in a floating condition while being filled with concrete introduced a building hazard of such serious nature as to render the adoption of the "Type B" method inadvisable. The ability to place nearly all concrete under favorable dry and open conditions, and the elimination from the boats of a considerable part of the longitudinal

reinforcing metal, which would have little value in the completed structure, are the principal advantages which were claimed for the "Type B" as compared with the "Type A" method.

Plate VI shows three progressive stages in the sinking of a boat and the building of a section of the dock, as proposed under both the "Type A" and the "Type B" methods.

Hollyday Plan.—On April 21st, 1914, R. C. Hollyday, M. Am. Soc. C. E., Civil Engineer, U. S. N., recommended the sectional boat method for the construction of the dock, in case it should be decided to depart from the original contract plans. Mr. Hollyday submitted outline details based on making each section or unit 50 ft. long, measured on the axis of the dock. The boat for each section would have a bottom of two layers of 12-in. square timbers bolted together, and fitted with detachable timber ends and side-walls; the ends and sides, designed for repeated use, would be extended upward to form the walls of a coffer-dam within which all concrete would be placed in the dry. The bottom of each section would be flat. The concrete floor of the dock would be 16 ft. thick, with steel reinforcement sufficient for all tensile stresses. The boat would be sunk to position as it was filled with concrete. The plan proposed to add considerable material to the concrete side-walls of the dock and to provide horizontal projecting shelves, 2 ft. wide, outside each side-wall at the elevation of the finished floor.

The Writer's Plans.—The addition of pig iron or metal scrap to concrete to increase the weight of the mass was recommended in July, 1913, to obviate the necessity for general changes in the plans in order to make the weight of the dock greater than any possible hydrostatic uplift and to hold the center line of pressures within the middle third of the concrete mass.

The substitution of steel plates and structural shapes for reinforced concrete was suggested in September, 1913, for use in making the boats or bases required for the sectional boat method of construction.

The method of construction and general plans subsequently adopted were submitted in April, 1914. These are described in detail later under the heading "Revised Plans".

Floating Type.—The desirability of providing a dry dock of the floating type, instead of the graving type, has been suggested. It is undoubtedly true that a dock of the floating type, having ample capacity, could be constructed in less time and at less cost than one of

the graving type; this statement is especially true for a dock at Pearl Harbor where the natural soil conditions render the deep excavation and sub-surface concrete work required for a graving dock both hazardous and very expensive. However, all questions as to the type of dock to be provided are in this case beyond discussion, inasmuch as the Congressional authorization for the work distinctly directs the construction of "one graving dry dock".

Revised Plans.

Revised plans for the dock, as shown in outline on Plate VII, were approved on November 19th, 1914, and its construction is now being energetically pushed in an effort to complete the work on or before June 30th, 1918.

General Details.—The dock will be built in sixteen sections, each covering the entire width and having a length in the direction of the dock axis of about 60 ft. The bottom of each section will be flat, and will consist of a structural steel frame embedded in concrete. The side-walls will be of concrete with a small quantity of vertical reinforcement near their outer faces. All concrete in each section will be placed under dry and open conditions favorable for performing concrete work, which will permit of continuous and thorough inspection. There will be a space between the bases of adjacent sections, diminishing from about 5 ft. wide at the bottom to 3 ft. wide at the top, which will be sealed in the bottom with concrete placed through a tremie; the upper part of the space between the bases will be filled with concrete placed in the dry after the dock is unwatered; the considerable distance between the bases is provided to facilitate the work of a diver in clearing the space before depositing the tremie concrete. The spaces between the side-walls of adjacent sections will be sealed with small coffer-dams, and the closure will be made with concrete placed under dry conditions after unwatering the dock. Reinforcing rods will be used to tie together the bases of the two outer sections.

The semicircular wall at the inner end of the dock will be made of large, pre-moulded, reinforced concrete, voussoir blocks in combination with concrete filling placed under dry and open conditions; a base similar in all respects to that used for the sections in the body of the dock will be used as a foundation for the end-wall; an accurately formed bed or seat will be prepared on the upper part of the base for the sup-

PLATE VII.
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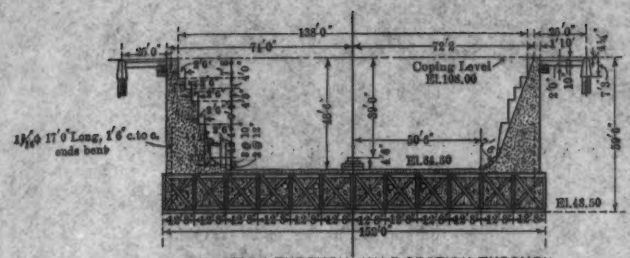
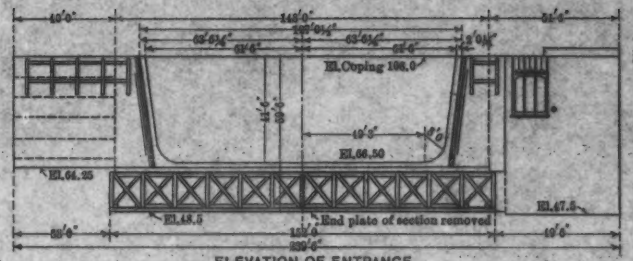
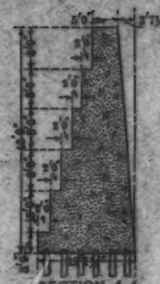
GENERAL REVISED OUTLINE PLAN OF THE DOCK



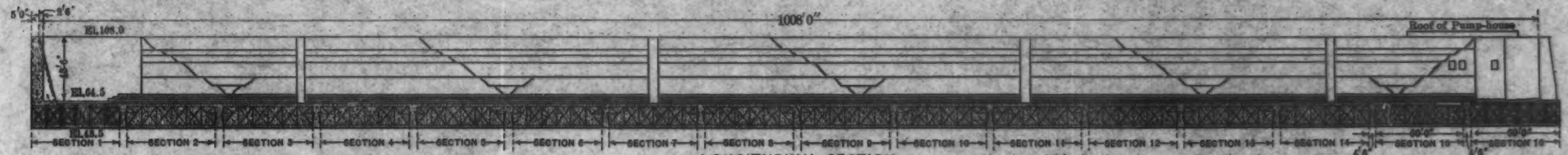
HALF-SECTION THROUGH BODY OF DOCK — HALF-SECTION THROUGH SLIDE



ELEVATION OF ENTRANCE

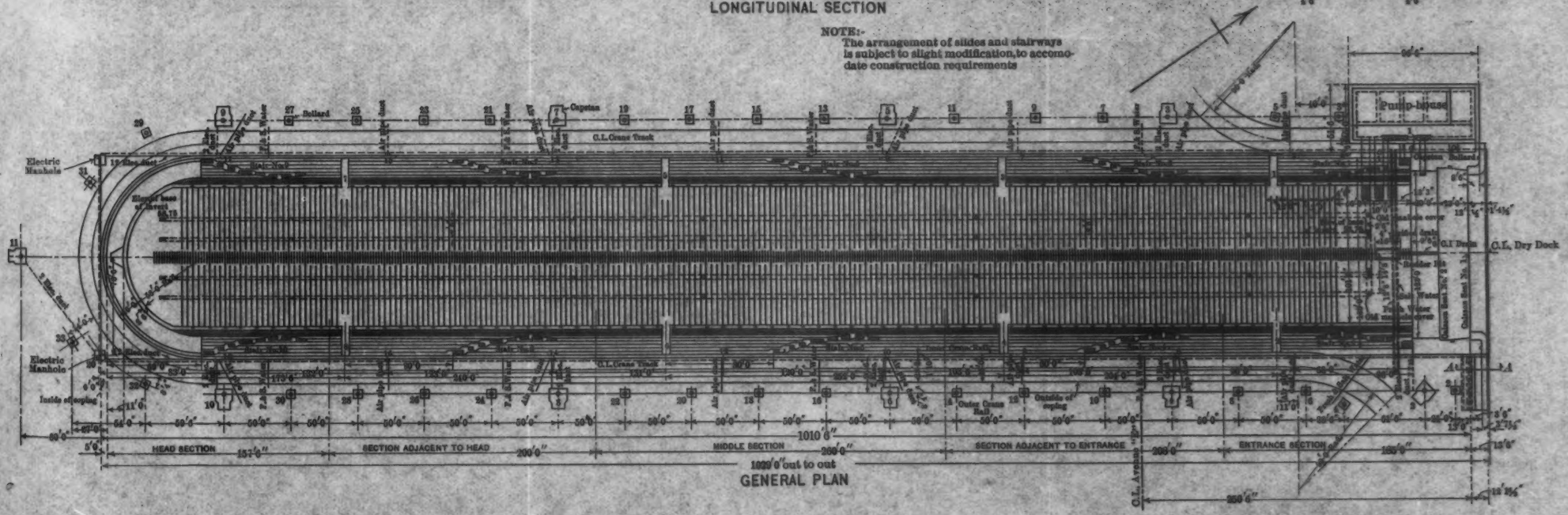


SECTION A-A
(On Enlarged Scale)



LONGITUDINAL SECTION

NOTE:-
The arrangement of slides and stairways is subject to slight modification, to accomodate construction requirements



GENERAL PLAN

port of the wall; the voussoir blocks will weigh about 150 tons each, and will be made in forms on the bank adjacent to the excavation, within reach of the Station's large floating crane, which will be used in setting the blocks; a space about 4 ft. wide will be left between adjacent voussoirs, and this will be closed by an inner and an outer coffer-dam, after which the space will be unwatered and filled with concrete; a keyway or grouting space will be provided between the bottom of each block and its bed, which will be sealed when the concrete is placed in the open space adjoining the block.

Additional concrete will be placed under the side-walls, to make the bottom of the bases flat, and additional piles will be driven to support this concrete. The pump-well will be near the entrance, and will be integral with the side-wall of the dock. The pump-well will be founded on a steel boat supporting a timber coffer-dam; the boat will be sunk by the addition of the concrete required in the construction of the pump-well. The concrete will generally be made much richer than originally required.

Stress Analyses.—The structural details are based on numerous stress computations, which in turn are based on various assumptions, as follows:

That maximum tide will reach an elevation of 102.5, which is 2.5 ft. above mean low tide and 1.3 ft. above mean high tide.

That sea water weighs 64 lb. per cu. ft.

That effective hydrostatic pressure on the bottom of the dock will equal full sea pressure at that depth, due to the extremely porous character of the soil.

That the earth side-banks of the excavation for the dock will take a natural slope of 1:1 both above and below tide level.

That the soil above extreme high tide level at Elevation 102.5 weighs 102 lb. per cu. ft., and that its submerged weight is 63 lb. per cu. ft. below this level. The equivalent liquid pressures on a vertical surface are 17.5 and 10.8 lb. per sq. ft., respectively.

That concrete weighs 150 lb. per cu. ft.

That there are transverse steel trusses embedded in the concrete bottom of the dock which provide the equivalent, per linear foot of dock, of 3 sq. in. of metal in the top chord and 1.6 sq. in. in the bottom chord.

That there is water pressure, but no earth pressure, on those portions of the upper horizontal surface of the base which project beyond the outer vertical faces of the side-walls.

That there is no Artesian pressure which would affect the hydrostatic pressure on the bottom of the dock.

On the foregoing assumptions, the dock was analyzed to determine the stresses which would result within its structure, depending on varying external forces and the manipulation of the dock. The stress diagrams on Plates VIII and IX have been made for the following cases:

Case A.—Uniform soil and water pressure under the bottom of the dock; dock empty; external water and earth pressure on side-walls.

Case B.—Uniform soil and water pressure under the bottom of the dock; dock empty; external water pressure only on side-walls.

Case C.—Uniform soil pressure under the bottom of the dock; dock full of water; external earth pressure on side-walls.

Case D.—Uniform soil pressure under the bottom of the dock; dock full of water; external water pressure only on side-walls.

Case E.—Uniform soil and water pressure under the bottom of the dock; ship load of 200 000 lb. per lin. ft. of dock, 120 000 lb. on keel block, and 40 000 lb. on each side-block; external water and earth pressure on side-walls.

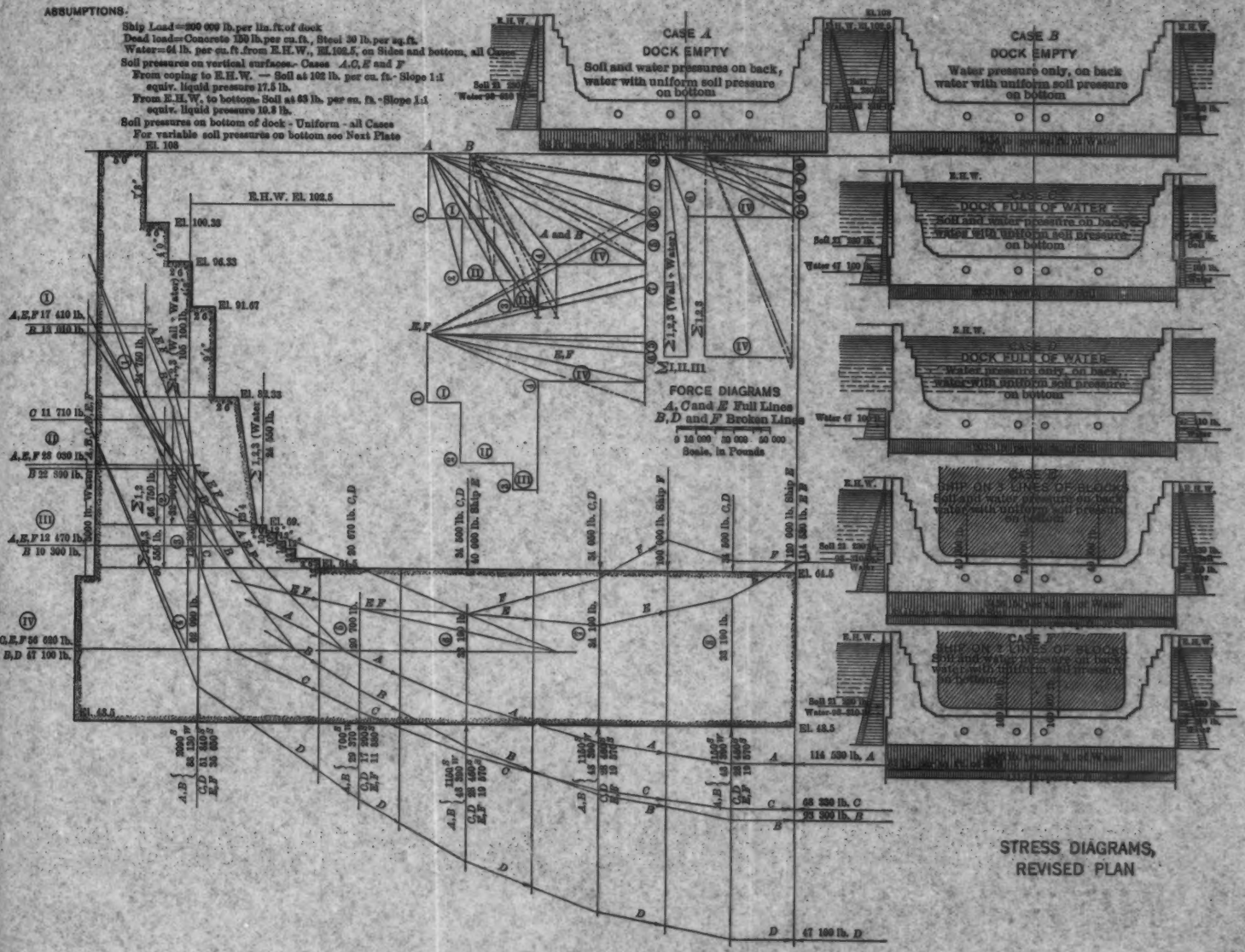
Case F.—Uniform soil and water pressure under the bottom of the dock; ship load of 200 000 lb. per lin. ft. of dock, one-half on each side-block; external water and earth pressure on side-walls.

Case G.—Intensity of soil pressure under the dock varies with the superimposed load and strain of dock; dock empty; external water and earth pressure on side-walls.

Case H.—Intensity of soil pressure under the dock varies with the superimposed load and strain of dock; dock empty; external water pressure only on side-walls.

Case I.—Intensity of soil pressure under the dock varies with the superimposed load and strain of dock; dock full of water; external water and earth pressure on side-walls.

Case J.—Intensity of soil pressure under the dock varies with the superimposed load and strain of dock; dock full of water; external water pressure only on side-walls.


ASSUMPTIONS.
Ship Load=800 000 lb. per lin. ft. of dock
Dead load=Concrete 150 lb. per cu. ft., Steel 30 lb. per sq. ft.
Water=64 lb. per cu. ft. from E.H.W., El. 102.5, on Sides and bottom, all Cases
Soil pressures on vertical surfaces.- Cases A, C, E and F
From coping to E.H.W. — Soil at 102 lb. per cu. ft. - Slope 1:1 equiv. liquid pressure 17.5 lb.
From E.H.W. to bottom- Soil at 63 lb. per cu. ft. - Slope 1:1 equiv. liquid pressure 10.8 lb.
Soil pressures on bottom of dock - Uniform - all Cases
For variable soil pressures on bottom see Next Plate
El. 108
CASE A
DOCK EMPTY
Soil and water pressures on back, water with uniform soil pressure on bottom
CASE B
DOCK EMPTY
Water pressure only, on back water with uniform soil pressure on bottom
CASE C
DOCK FULL OF WATER
Soil and water pressure on back, water with uniform soil pressure on bottom
CASE D
DOCK FULL OF WATER
Water pressure only, on back, water with uniform soil pressure on bottom
CASE E
SHIP ON 7 LINES OF BLOCKS
Soil and water pressure on back, water with uniform soil pressure on bottom
CASE F
SHIP ON 7 LINES OF BLOCKS
Soil and water pressure on back, water with uniform soil pressure on bottom
R.H.W. El. 102.5
El. 100.33
El. 96.33
El. 91.67
El. 88.33
El. 69.
El. 64.5
El. 48.5
A and B
E, F
FORCE DIAGRAMS
A, C and E Full Lines
B, D and F Broken Lines
0 10 000 20 000 30 000
Scale, in Pounds
A, E, F 17 410 lb.
B 13 810 lb.
C 11 710 lb.
A, E, F 25 030 lb.
B 22 890 lb.
A, E, F 12 470 lb.
B 10 300 lb.
A, C, E, F 56 600 lb.
B, D 47 100 lb.
114 530 lb. A
68 230 lb. C
73 300 lb. E
47 100 lb. D
STRESS DIAGRAMS,
REVISED PLAN

Case K.—Intensity of soil pressure under the dock varies with the superimposed load and strain of dock; ship load of 200 000 lb. per lin. ft. of dock, 120 000 lb. on keel block, and 40 000 lb. on each side-block; water and earth pressure on side-walls.

Case L.—Intensity of soil pressure under the dock varies with the superimposed load and strain of dock; ship load of 200 000 lb. per lin. ft. of dock, one-half on each side-block; external water and earth pressure on side-walls.

The load per square foot on the soil underlying the dock is: for Case *A*, 82 lb.; Case *B*, 82 lb.; Case *C*, 2 033 lb.; Case *D*, 2 033 lb.; Case *E*, 1 398 lb.; Case *F*, 1 398 lb.; Case *G*, 82 lb. under the sides, increasing to 217 lb. at a distance of 50.5 ft. from the center, then decreasing to zero at a point 28 ft. from the center; Case *H*, same as Case *G;* Case *I*, 2 350 lb. under the sides and to a point 50.5 ft. from the center, then uniformly decreasing to 1 395 lb. at the center; Case *J*, same as Case *I;* Case *K*, 82 lb. at the sides, increasing uniformly to 2 007 lb. at a point 28 ft. from the center, then uniform to the center; Case *L*, 82 lb. under the sides, increasing uniformly to 2 304 lb. at a point 14 ft. from the center, then uniform to the center.

The pressure on the soil is so small when the dock is empty, as treated in Cases *A* and *B*—only 82 lb. per sq. ft.—that it seems improbable that there will be any great variation in the intensity of the pressure. Assuming, however, that there will be a variation, as given under Cases *G* and *H*, there would be a reduction in the bending moment at the center of the dock, which, in comparing Case *G* with Case *A*, would reduce the stresses in the steel and concrete from 15 430 and 472 lb. per sq. in., respectively, to 12 150 and 412 lb.; and, comparing Case *H* with Case *B*, would reduce the unit stress from 25 330 and 592 lb. to 23 260 and 558 lb. These latter stresses, as computed for Cases *B* and *H*, cannot actually occur, as the specification governing the work requires that the back-filling outside the side-walls of the dock shall be placed before the dock is emptied.

The slight variation in soil pressure beneath the dock, computed for Cases *I* and *J*, as compared with the condition of uniform pressure computed for Cases *C* and *D*, results in reducing the bending moment at the center of the dock. Comparing Case *I* with Case *C:* this reduces the unit stresses in the steel and concrete from 17 300 and 413 lb. to 5 480 and 213 lb. Comparing Case *J* with Case *D:* the unit stresses

in the steel and concrete are reduced from 28 350 and 535 lb. to 15 080 and 334 lb. The unit stresses would be still further reduced with further small variations in the intensity of the supporting soil pressure. Inasmuch as a considerable deformation or strain of the dry dock structure would necessarily have to occur before destructive unit stresses could be developed, and as these changes would assuredly introduce variations in the intensity of the soil pressure, with reductions in stresses even greater than those previously computed, it may be concluded that the dock as designed will be safe when filled with water.

Slight variations in the intensity of the soil pressure beneath the dock, to conform with the variations in the superimposed load when the dock is pumped out and carries a ship on the blocks, result in comparatively large changes in the unit stresses in the dock structure. Assuming variations in the soil pressures, which would logically follow from the more or less elastic nature of the dock structure when under strain, and with variations confined within the narrow limits computed for Cases *K* and *L*, as compared with the uniform pressures computed for Cases *E* and *F*, shows that the unit stress in the steel in the bottom chord would be reduced from 14 140 lb. tension in Case *E* to 750 lb. compression for Case *K*, with corresponding reduction in the unit compression in the concrete from 386 to 50 lb.; comparing Case *L* with Case *F*, the unit stress in the bottom chord steel is changed from 14 140 lb. tension to 116 lb. compression, and the unit compression in the concrete is reduced from 386 to 80 lb. From these figures it is concluded that the dock can safely carry the assumed ship loading.

It is apparent from the foregoing analysis that the critical condition of loading is that shown by the stress diagram, *A*, which assumes the dock to be empty, with hydrostatic and earth pressures on the side-walls. Slight variations in the intensity of the soil pressure beneath the dock, for this condition, will have very little effect on the unit stresses in the dock structure. The center line of pressure within the bottom of the dock prism may be moved farther into the body of concrete by assuming an induced passive soil pressure or greater active soil pressure on the side-walls, but this would bring the center line of pressure dangerously near the haunch or toe of the side-walls; vertical reinforcement in each side-wall, equivalent to 0.9 sq. in. of steel per lin. ft. of dock, has been provided to meet this contingency.

PLATE IX.
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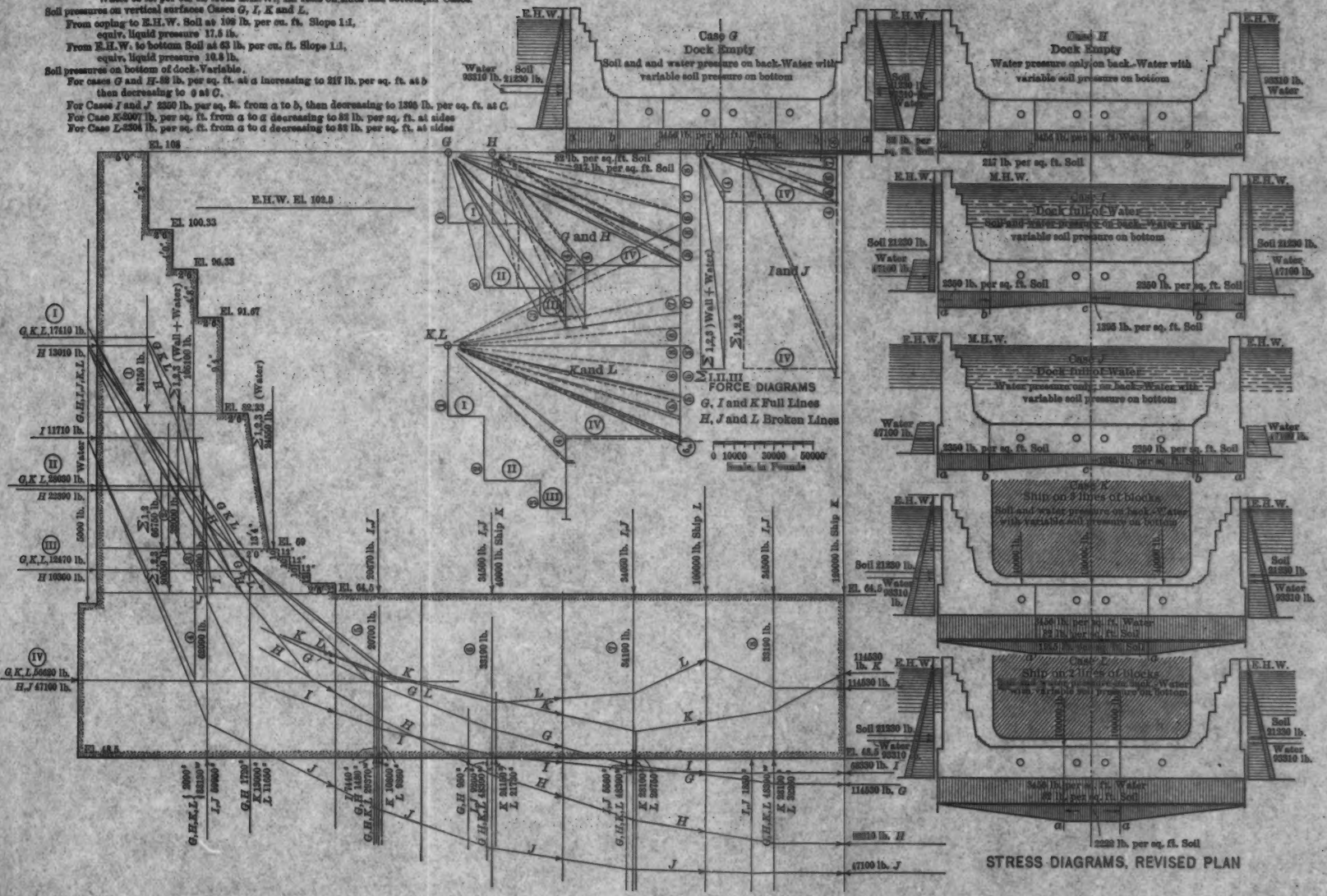
ASSUMPTIONS: Ship load = 250 000 lb. per lin. ft. of Dock
Dead load = Concrete 150 lb. per cu. ft., Steel 30 lb. per sq. ft. bottom.
Water 64 lb. per cu. ft. from E.H.W., El. 102.5 on sides and bottom, all Cases.
Soil pressures on vertical surfaces Cases *G*, *I*, *K* and *L*.
From coping to E.H.W. Soil at 106 lb. per cu. ft. Slope 1:1, equiv. liquid pressure 17.5 lb.
From E.H.W. to bottom Soil at 63 lb. per cu. ft. Slope 1:1, equiv. liquid pressure 10.5 lb.
Soil pressures on bottom of dock-Variable.
For cases *G* and *H*-82 lb. per sq. ft. at *a* increasing to 217 lb. per sq. ft. at *b* then decreasing to 0 at *C*.
For Cases *I* and *J* 2350 lb. per sq. ft. from *a* to *b*, then decreasing to 1395 lb. per sq. ft. at *C*.
For Case *K*-2007 lb. per sq. ft. from *a* to *d* decreasing to 82 lb. per sq. ft. at sides
For Case *L*-2504 lb. per sq. ft. from *a* to *d* decreasing to 82 lb. per sq. ft. at sides



STRESS DIAGRAMS, REVISED PLAN

The weight of the dock is 12 480 lb. per lin. ft. of dock in excess of the hydrostatic pressure on the bottom.

The safety and sufficiency of the dock structure are further insured by various forces which have not been evaluated in the foregoing treatment, but which have material magnitude and will be positive and reliable in their action. These additional elements of strength are in the nature of factors of safety, and include the following items:

(*a*).—The pile-supported concrete prism, which surrounds the dock to carry the outer rail of the crane track, is secured to the side-walls of the dock near the coping level by closely placed reinforced concrete ties, and serves to anchor or support the side-walls against inward movement.

(*b*).—The "plums" or "one-man stone", which will be embedded in the concrete, weigh from 167 to 173 lb. per cu. ft. The unit weight of 150 lb. assumed for concrete in making the stress diagrams was not increased on account of these "plums". The specifications provide that as large a percentage of plums may be used as may be consistent with the obtaining of strong, dense, and impervious concrete. The addition of plums will undoubtedly add quite a little to the assumed average unit weight of the mass concrete.

(*c*).—It has just been shown that the critical loading condition for the dock is that discussed under Case *A*, which assumes the dock to be empty. Under this loading condition, the weight of the blocking equipment which will be secured to the floor of the dock will be directly effective, and will reduce materially the unit stresses in the dock floor.

(*d*).—The weight of the soil on the shelves which project beyond the outer faces of the side-walls will add to the weight of the dock to resist flotation in the structure.

Special Plant.—The contractors are providing two special items of plant which are necessary for carrying on the work. These consist of a floating dry dock and a steel structure which, for want of a better name, will be called a coffer-dam-boat.

The floating dock is being made of timber, and will have a lifting capacity of about 3 500 tons. The general dimensions of the dock are: over-all width, 100 ft.; clear width between side-walls, 76 ft.; depth of bottom pontoons below floor, 12 ft.; over-all length, 214 ft.; height of side-walls above floor of dock, 34 ft.; top width of side-walls, 8 ft.;

length of side-walls, 164 ft. The bottom is divided into five pontoons, each 32 ft. wide, with a space 12 in. wide between adjacent pontoons. There is an outrigger, 25 ft. long, at each end of the dock. Each pontoon has a small side-wall, 4 ft. high and 12 ft. wide, at each end, to which the main side-walls of the dock are secured. The maximum draft of the floating dock is 41.5 ft., and will occur when the dock is lowered to receive the coffer-dam-boat; the top of the floating dock will be 4.5 ft. above the water surface in this position. There is a bracket on the inner face and near the top of each side-wall which will be loaded with about 100 tons of ballast to relieve the bending moment in the pontoons under certain conditions of loading.

The coffer-dam-boat is a structure which fulfills several requirements; it is a true coffer-dam, as it will exclude water from a working space which is open to the atmosphere and is below sea level; it is a boat, because it will be used for transporting and for containing a load, and because it can be alternately sunk and floated. Its service as a boat will be actually required to transport the partly completed bases for the sections of the concrete dock and to contain water which will be used to increase its weight; it will be repeatedly sunk, to locate the bases on the bottom and to impose its loaded weight as ballast on the bases to resist hydrostatic uplift; it will be repeatedly floated, to clear the completed dock sections of falsework and for repeated use in handling the bases on which the sections are founded. The structure will be entirely of steel, with the exception of a small quantity of removable timber bracing in the end pontoons, and may be described as an open boat, 108 ft. long, 60 ft. wide, and 38.5 ft. deep, with sides extended longitudinally and downward to form a U-shaped coffer-dam which will surround the ends and bottom of the boat. The coffer-dam space at each end of the boat is 22 ft. wide, and is 6 ft. 3§ in. high under the boat. There is permanent steel bracing, together with removable timber bracing, in each vertical leg of the coffer-dam. Numerous braced steel stools are connected to the bottom of the boat, for use while transporting a base and to support the weight on the bottom when the boat is sunk and resting on a base frame. The interior space in the boat is sub-divided into four equal water-tight compartments by a transverse and a longitudinal bulkhead; these compartments are fitted with pumps, piping, and valves for use in adjusting the trim of the boat; the bulkheads are not heavily braced or stiffened, as only a

small difference in level of the water in the four compartments is anticipated. Figs. 13 and 14 are photographs of a model showing the general form and construction of the coffer-dam-boat and giving some idea of the heavy framing and bracing required. The estimated weight of the structure is 875 tons.

Field Work.—A sufficient area underlying the dock will first be cleared of all earth, débris, and pile heads to a level sub-grade 12 in. below the finished grade. Shortly before placing the base for any section of the dock, the area which is to be covered will be swept by a frame having its lower edge held at a level 6 in. below the finished grade, to be sure that all projecting points have been removed, after which the area will be carefully brought to the required grade with broken stone and sand filling.

All piles which have been driven and were injured by the previous upheaval, or which will be too short to be reliable after the excavation is deepened, will be replaced with suitable piling. The additional piling required for the supports of the heavier dock will also be driven.

The structural steel frame required for the base of each section will weigh about 153 tons. This steelwork is being fabricated at Ambridge, Pa., in accordance with the standard specifications for material and workmanship contained in the 1914 handbook of the United States Steel Products Company for steelwork for buildings, using steel covered by Classes "B" and "C". Fig. 10 is a photograph of a model of the steel frame for one of the bases, and Fig. 11 shows the base as it will appear when erected on the floor of a floating dry dock. It may be noted that the model of the floating dock shown on Fig. 11 and on any subsequent illustrations is that of an assumed dock, and does not represent the one now under construction and previously described.

The steel frame comprises seven trusses, 152 ft. long, and 10 ft. from center to center; the trusses will be in a direction transverse to the axis of the dock when in position. Each truss has twelve panels and is 14.75 ft. deep over all. Adjacent trusses will be connected by cross-bracing at each panel point. The top chords of the interior trusses in the frame at the center panels are composed of two 6 by 6 by $\frac{5}{8}$-in. angles, one 20 by $\frac{1}{2}$-in. plate, and two $14\frac{1}{2}$ by $\frac{3}{8}$-in. side-plates; the lower chords of the interior trusses are composed of two 6 by 4 by $\frac{9}{16}$-in. angles and one $14\frac{1}{2}$ by $\frac{3}{8}$-in. plate. The intermediate posts in the interior trusses are composed of four 6 by 4 by $\frac{9}{16}$-in. angles and one

12 by ½-in. plate, and are very heavy, in order to carry the water load of the boat when the base is first settled into position. The exterior trusses of each frame and the end bracing connecting the seven trusses in a frame have no diagonal web members; the top chords are made of a 15-in. 33-lb. channel, with the web horizontal, in combination with a vertical 30 by $\frac{5}{16}$-in. web-plate, which is stiffened near the lower edge with a 3 by 3 by $\frac{5}{16}$-in. angle; the angles in the lower chords have their vertical legs turned down, in order that the outstanding legs will be more effectively protected by the concrete matrix in which the frame is to be embedded. The steel frames will be erected, aligned, and riveted on the floor of the floating dock.

The concrete work may be separated into two principal classes; the first will include practically all concrete which is placed under dry and favorable conditions, and the second will include that placed under water through a tremie. About 133 700 cu. yd. of concrete will be placed under dry conditions, including the massive work in the body of the dock, in the pump-well, and in the portions of the quay walls adjacent to the dock entrance; this concrete will be mixed in the proportion of 100 lb. of cement, ¾ cu. ft. of coral sand, 1½ cu. ft. of crusher screenings, and 4 cu. ft. of broken stone; "plums" or large pieces of rock may be embedded in the concrete in quantity which will be consistent with obtaining satisfactorily dense, strong, and impervious concrete. About 6 500 cu. yd. of concrete will be placed under water through a tremie, which includes about 2 500 cu. yd. for making the seals between the bases of adjacent sections and under the voussoirs at the head of the dock and 4 000 cu. yd. in the pump-well; this concrete will be mixed in the proportion of 100 lb. of cement, ⅔ cu. ft. of coast sand, 1⅓ cu. ft. of crusher screenings, and 3½ cu. ft. of broken stone. The exposed faces of the side-walls and the wearing surface of the floor of the dock, also the interior walls and floors of the pump-well and connected culverts, will be finished with a coating of "lavalithic", which will be placed at the same time as the concrete backing; the "lavalithic" will be mixed in the proportion of 100 lb. of cement, ½ cu. ft. of coast sand, and 1½ cu. ft. of crusher screenings; and the coating will be from 1½ to 2 in. thick.

Work will be started placing the concrete to embed a steel base frame just as soon as the erection of the steel frame is completed. A layer of thin boards or other suitable material will be placed on the

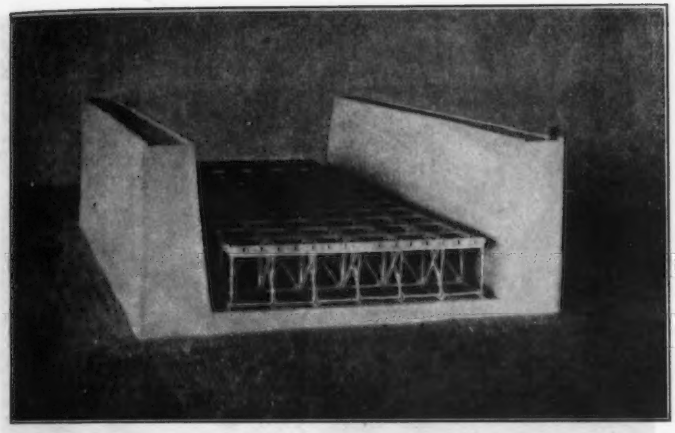


FIG. 11.—STEEL FRAME FOR BASE OF SECTION ERECTED ON FLOATING DOCK.

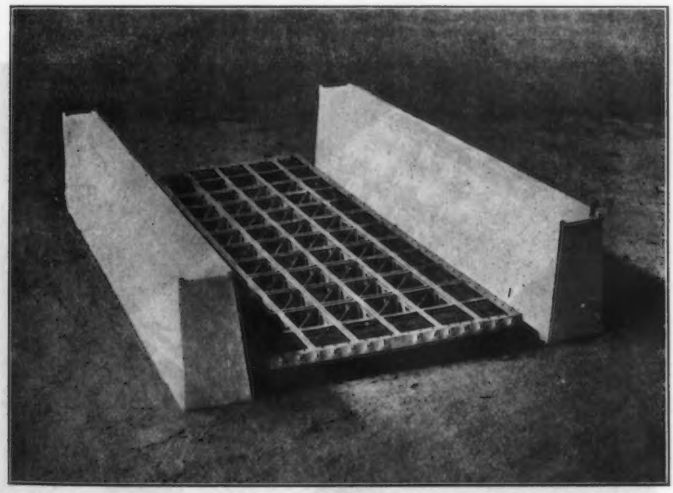
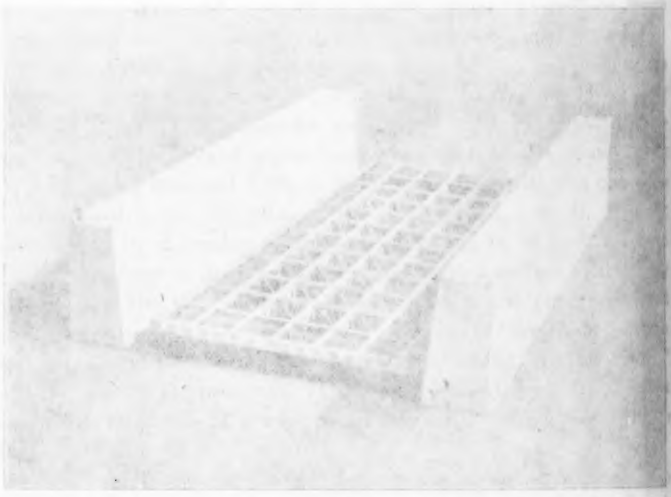


FIG. 12.—STEEL FRAME WITH EMBEDDING CONCRETE PARTLY PLACED.
(At this stage the partly completed base is ready to be removed from the floating dock.)



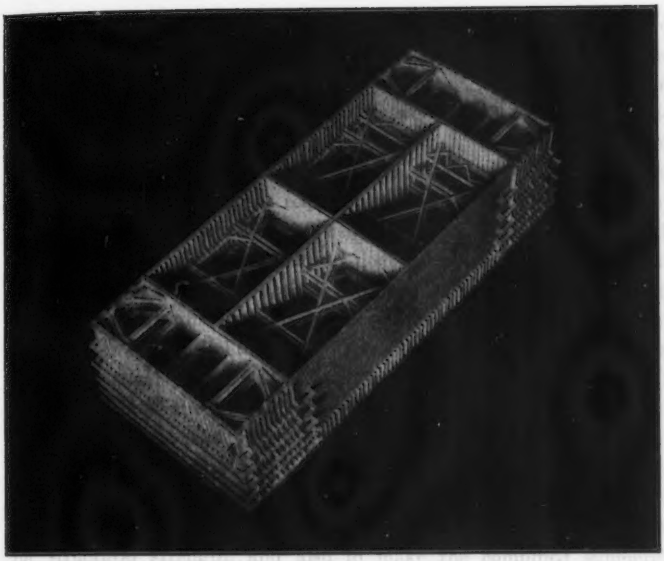


FIG. 13.—TOP VIEW OF COFFER-DAM-BOAT USED FOR TRANSPORTING AND SINKING SECTION BASES, AND REQUIRED FOR THE COMPLETION OF EACH SECTION.

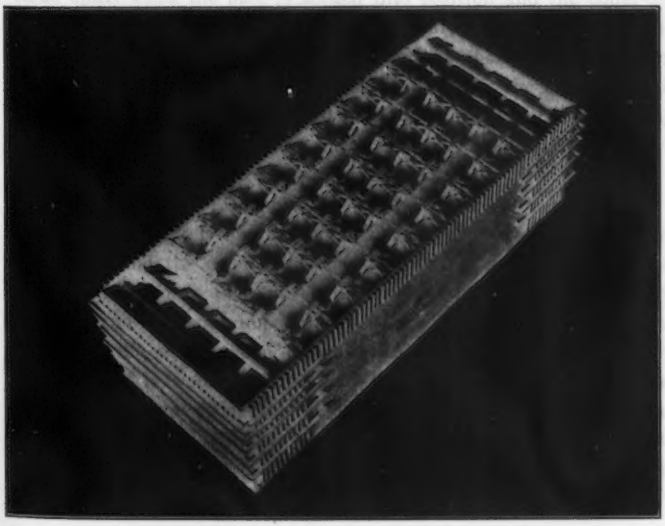


FIG. 14.—BOTTOM VIEW OF COFFER-DAM-BOAT USED FOR TRANSPORTING AND SINKING SECTION BASES, AND REQUIRED FOR THE COMPLETION OF EACH SECTION.

FIG. 13.—TOP VIEW OF COPPER-DAM-BOAT USED FOR TRANSPORTING AND SINKING SECTION BARGE, AND REQUIRED FOR THE COMPLETION OF EACH SECTION.

FIG. 14.—BOTTOM VIEW OF COPPER DAM-BOAT USED FOR TRANSPORTING AND SINKING SECTION BARGE, AND REQUIRED FOR THE COMPLETION OF EACH SECTION.

floor of the floating dock before the steel frame is erected and on which the concrete will be placed; this loose covering will protect the dock floor from injury when the concrete is placed, and will tend to prevent the suction or adhesion between the base and the floor of the floating dock when the base is removed. The steel frame and side-walls of the floating dock will provide ready means for securing and shoring the form work required to confine the concrete. About 3 500 cu. yd. of concrete will be placed for each base while the structure is carried on the floating dock; this concrete will be about 8 ft. thick over the entire area of the base, and will be banked solidly around the side-walls and ends to the extreme top of the steel frame; care will be taken to obtain a tight seal between the banked concrete and the vertical 30-in. plate which forms part of the top chord of the side girders and end bracing of the steel frame; the thickness of the concrete banks will be made ample to prevent any inward movement of the sides and ends of the base when subjected to the pressures which will occur during the later stages of the work; the depth of 8 ft. of concrete in the bottom of the base is required for structural strength, and also to make the combined submerged weight of the base and coffer-dam-boat sufficient to resist the hydrostatic uplift when the base is settled into position and the coffer-dam unwatered. The estimated weight of the partly completed base when ready to be removed from the floating dock is about 7 250 short tons; the estimated displacement of the base, assuming the dished interior to be free of water, is about 4 320 short tons. Fig. 12 is a photograph showing the appearance of a partly embedded steel frame resting on the floating dock and ready to be removed.

After completing that portion of the work on a base which is performed while it is on the floor of the floating dock, the dock will be lowered and the coffer-dam-boat floated to position over the base. The floating dock is then raised until it has lifted the joint between the base and the superimposed coffer-dam-boat to a level above the surface of the water, when the joint will be made with bolts, using a lead gasket between the top flange of the base and the lower flange of the coffer-dam to obtain water-tightness. The stools under the bottom of the boat will bear over panel points of the trusses in the steel frame when the coffer-dam-boat and base are connected, and the stools and frame will be connected with bolts.

The top chords of the outside trusses and end girders of the steel frames are fitted with exterior shelf-angles forming pockets to receive the lower ends of bolts which will be used to bond together the coffer-dam-boat and bases; these bolts will be long enough to reach the top of the coffer-dam to facilitate their detachment after completing a section of the dock without requiring the services of a diver. The bolts connecting the stools combined with the exterior bolts will have ample strength to carry the weight of a suspended base. Fig. 15 shows the coffer-dam-boat in position over a base with the floating dock raised to bring the joint between the coffer-dam and the base above the surface of the water. Fig. 16 shows the floating dock lowered sufficiently to float the coffer-dam-boat with its suspended base or load which is ready to be transported to the desired location over the dock site. All the water entering the partly completed dish-shaped base when the floating dock is lowered to receive the coffer-dam-boat will be pumped out after the joint between the coffer-dam and base has been completed, and as this space will then be fully protected against the entrance of water, by the walls of the coffer-dam with its temporary concrete steel bottom, there should be no difficulty in keeping the space dry during the transportation and sinking of the base and while the concrete is being placed to complete the dock section of which the base will form a part.

When the coffer-dam-boat with suspended base has been floated to the desired location, the boat will be sunk by admitting water into its interior compartments until the base comes to rest on the bottom of the dock excavation. The rate of sinking and trim of the boat will be under perfect control during the entire sinking process, and, by regulating the quantity of water in the boat, the base could readily be raised and again sunk if it should be desired to clear the bottom or to obtain a better location. After the base has been satisfactorily placed, it will be firmly held in position by the weight of the coffer-dam-boat aided by the weight of the water in the boat. Fig. 17 shows the coffer-dam-boat resting over a base which has been placed; the boat is full of water but the space within the coffer-dam is dry. The concrete required to complete the section to a point above high-tide level will then be placed in the dry space within the coffer-dam; the temporary bracing in the ends of the coffer-dam will be modified as required for the construction of the side-walls. The

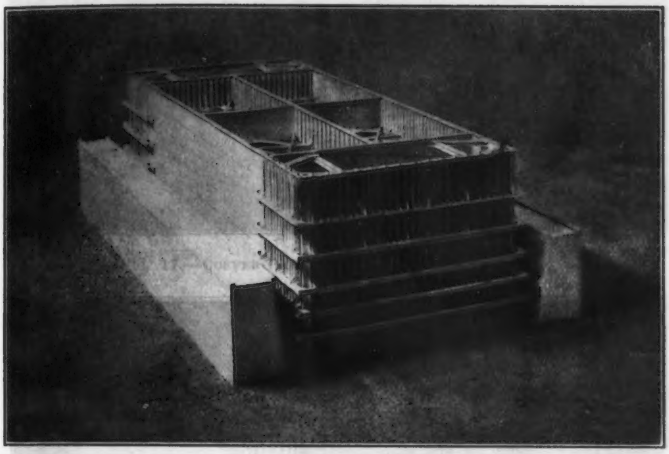


FIG. 15.—COFFER-DAM-BOAT MOUNTED ON SECTION BASE, READY FOR SINKING THE FLOATING DOCK.

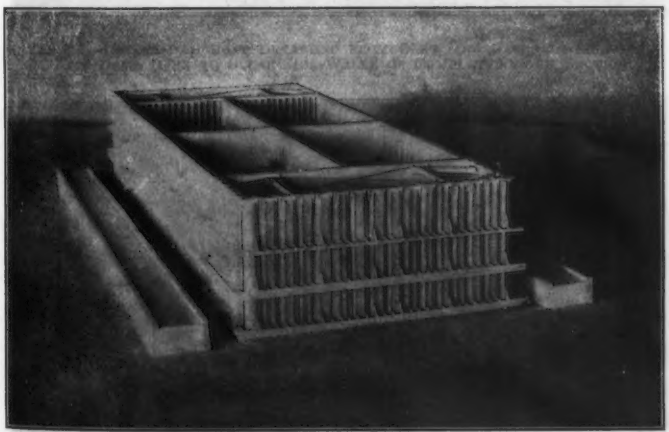
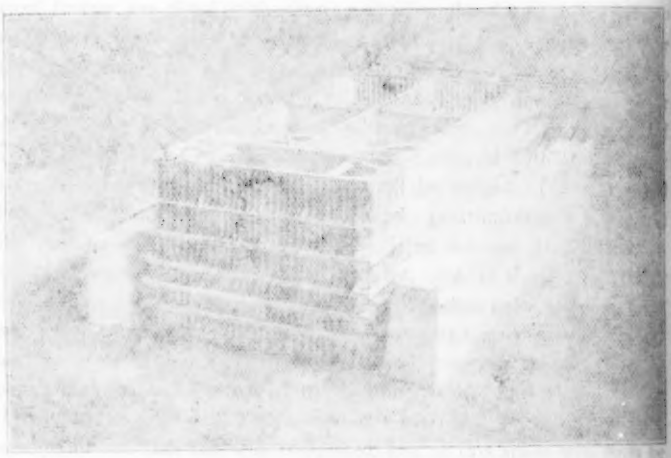


FIG. 16.—FLOATING DOCK IN DEEPEST POSITION, JUST PREVIOUS TO REMOVAL OF COFFER-DAM-BOAT WITH ITS ATTACHED BASE.



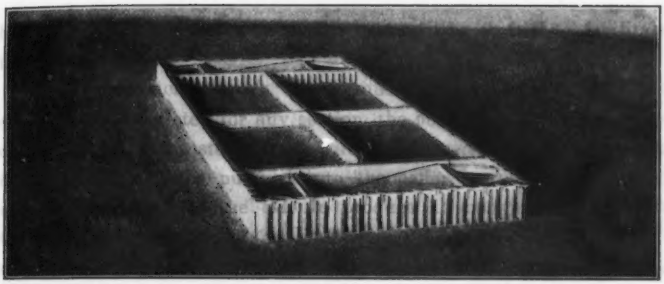


FIG. 17.—COFFER-DAM-BOAT IN FLOODED CONDITION.

FIG. 18.—COFFER-DAM-BOAT DETACHED FROM BASE, AND GATES IN SIDES OF COFFER-DAMS OPEN TO CLEAR SIDE-WALLS OF COMPLETED DOCK SECTION.

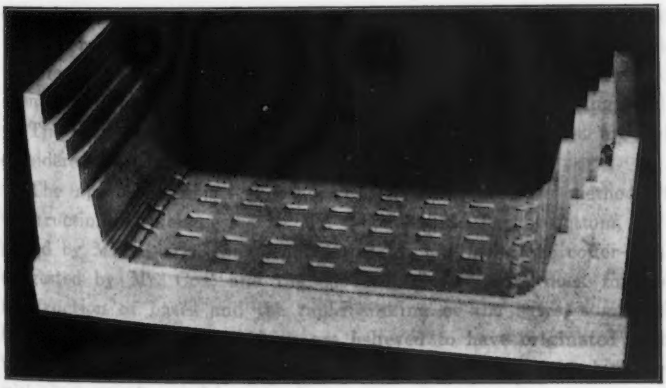
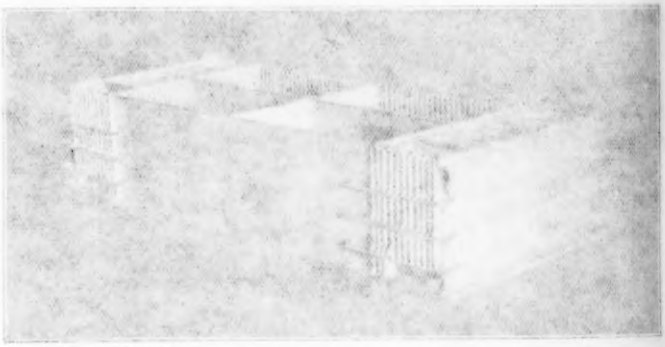
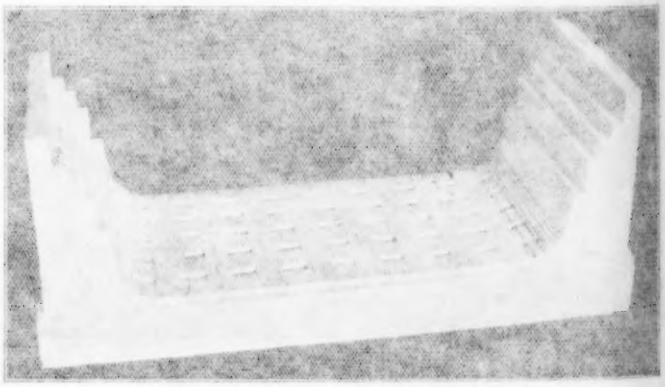


FIG. 19.—ISOMETRIC VIEW OF COMPLETED SECTION OF DOCK.





bracing between the stools under the boat will be shifted to facilitate the concrete work.

When the concrete work which must be performed within the coffer-dam is completed for any section, the bolts connecting the stools of the boat with the steel frame will be removed and the coffer-dam flooded; water will be pumped into the coffer-dam space to a level a little above that of the tide to produce an outward pressure on the walls of the coffer-dam to liberate any temporary bracing which may have been placed between the concrete and the walls of the coffer-dam; the exterior bolts will then be removed and the water pumped out of the boat; when the boat is practically dry the side-gates in the ends of the coffer-dam will be removed to enable the coffer-dam-boat to be floated over the concrete side-walls of the completed section. Fig. 18 shows the coffer-dam-boat floating at minimum draft, with the coffer-dam gates opened ready for the coffer-dam-boat to be removed from a completed section to repeat the cycle of operations.

Fig. 19 shows an isometric projection of a completed section. The rectangular spaces in the floor are the spaces in which the stools rested; they will be filled with concrete when the dock is unwatered. The concrete below the surface of the floor to the top of the steel frame is 15 in. thick.

The semicircular wall at the head of the dock will be built on a bed which will be prepared within the steel coffer-dam prior to the detachment of the coffer-dam-boat from the base for the inner section.

After completing the sixteen sections of the dock, and the pump-well, the open spaces between the sections and the well will be sealed with tremie-placed concrete or small coffer-dams, and the dock unwatered. The open spaces will then be completed under dry and open conditions, and the side-walls will be carried to the coping level.

The portions of the work which have not been described are not considered to involve any special construction difficulties.

The approved design incorporates the sectional-boat method of construction, first suggested by Mr. Cox in 1911, the flat bottom, suggested by Mr. Harris, the use of water for weighting the coffer-dam, suggested by Mr. Cox, and the use of a floating dry dock for the construction of bases and the rapid sinking of the bases, suggested by Mr. Noble. Features which are believed to have originated with the writer include the use of structural steel frames in combination with concrete for making the bases, instead of reinforced concrete;

the transporting or carrying device to insure the bases against sinking or loss from the time they are released from the floating dock until they are finally settled in position in the bottom of the dock excavation; the facilities for readily adjusting the draft and trim of the bases while being moved to position and being settled; the provision which enables the repeated settling and raising and settling of any base, as may be desired, to secure accurate alignment and satisfactory bed; the elimination of all timber sheeting and nearly all timber bracing required for construction of the dock sections, with large reduction in cost and building hazard; the elimination of practically all tremie-placed concrete without involving endangering conditions; and the use of voussoir blocks in building the semicircular wall at the inner end of the dock.

The elimination of tremie concrete is considered a most valuable feature of the approved plans, inasmuch as there can be no absolute certainty that concrete deposited under water will possess the uniform degree of perfection which is essential if the dock is to be satisfactory when completed. Although other methods which have been suggested would obviate the need for tremie concrete, the results would be accomplished at the expense of serious building hazards involved by long periods of flotation while unwieldy boats of more or less temporary construction were being loaded with large masses of concrete, and by the repeated construction and use of large timber coffer-dams. The finished details of the plans represent the combined studies of the several members of the Corps of Civil Engineers who have worked on the problem, the contractors' engineers, and the Bureau's technical assistants.

Conclusion.

The writer takes pleasure in mentioning the valued assistance rendered him in the preparation of this paper by Mr. E. R. Gayler, and Mr. Joseph Michaelson. The record would not be complete, however, without special tribute to the late Alfred Noble. He should be given the credit for final success in bringing about a resumption of work under most amicable arrangements between the contractors and the Government, in accordance with plans which apparently provide the greatest practicable safeguards in the expenditure of the country's funds with every promise of the ultimate satisfactory completion of the greatest public work of the United States Navy.

DISCUSSION

Mr. Fowler.

CHARLES EVAN FOWLER,* M. AM. SOC. C. E. (by letter).—The Profession is greatly indebted to Mr. Stanford for his comprehensive paper on the Pearl Harbor Dry Dock.

The construction of graving docks in the United States still remains very much of a novelty, although abroad it is a very common occurrence. The very fact of their novelty, although making the paper of greater value, is undoubtedly one of the causes of trouble in their construction, as neither the Government nor the contractors are able to obtain engineers or workmen who are experienced in their design or construction.

The sites for naval dry docks are seldom chosen by the designers, except within very narrow limits, but are selected for strategic, political, or other evident reasons, entirely aside from their suitability for a structure of such magnitude. Then, again, the time allowed for designing is usually all too short for a comprehensive study of the conditions which will be encountered, and must be taken into account in both the design and construction.

Imagine, too, the slight chance which bidders have to study carefully and digest the problem in the short time provided by the law for preparing and submitting bids. The bidder, of course, is in no way obligated to verify the borings, which, if made by the wash-boring process, are almost always unreliable and likely to suggest conditions which properly made core borings would show to be non-existent.

The construction of graving docks in England is very common in connection with projects for immense wet docks, and, in some cases, they are constructed parallel to the lock entrances to the wet dock, as at Tilbury, so that in case of an accident to the lock, they can be used as entrances to the wet dock. Comprising as they do a portion of such a great system of construction, it is natural that more time should be afforded in which to study the conditions and design, and consequently less trouble would be encountered during construction.

After reading Mr. Stanford's paper, the writer can see no reason for changing the conclusions reached some years ago, that the proper method of constructing a dry dock where conditions are unfavorable, as was the case at Mare Island and at Pearl Harbor, would be to do it under water, which was done successfully at Kobé, Japan, and has been fully described† by the writer.

That there may be serious objections to such a method is granted, but they are not such as cannot be readily overcome by the careful designer and constructor. The paramount objection is that of obtaining

* Seattle, Wash.

† "Sub-Aqueous Foundations," Chapter XXX.

Mr. Fowler.

good results in depositing concrete in sea water, and the writer's experience leads him to believe that, if made in this way, with proper equipment and care, the concrete will be just as good as, or better than, if deposited in the dry, and at no increase in cost.

The method used should be that known as the "Detroit Tunnel Method", and the tremie should be kept buried in the concrete at all times and moved around slowly, so that the concrete does not have to run to any great distance and does not come to any extent into contact with the water. This will obviate the washing of the cement, laitance, and consequent poor concrete.

Strange to record, perhaps, the writer's experience has shown that a smaller pipe for the tremie than any of those used at Pearl Harbor has given better results. That used at Tacoma for depositing concrete in depths between 50 to 60 ft. was of 10-in. lap-welded, screw-jointed pipe. This was successful because the interior was smooth, the joints allowed no water to leak in, and the tremie was light enough to allow it to be moved easily and properly. This work has been described by the writer,* who has demonstrated that riveted pipe is very objectionable.

Where riveted pipe has been used, it has always been larger and has had to be kept filled above water level, which is unnecessary with the smooth, screw-jointed pipe. For shallow depths, a square wooden pipe, with its smooth interior, will be found to be very good.

Although it is undoubtedly best to use a 1:2:4 concrete for under-water work, one of 1:3:5 will be found to give good results with a proper tremie and with proper care.

A few years ago, and simultaneously with its use at Kobé, the writer proposed the under-water method for a large dry dock in a mud foundation, with piling cut off under water, believing it to be the surest way of preventing trouble due to upward pressure. Practically all the weight is available when the dock is pumped out, the anchorage of the piling is effective, and a steel mat of reinforcing bars can be placed in under-water concrete, if necessary, to resist the arch action in a wide dock.

Experience has shown† that where piles are driven with a cast-steel follower cap to which their heads are fitted, they are left in better shape than when cut off under water.

The box type of coffer-dam was proposed for the case mentioned, such as that used at Mare Island and that first used at Pearl Harbor, but the very great care necessary to build a fairly tight dam and the danger of just such a failure as was realized at Pearl Harbor, would seem to make it a grave question whether its use is ever advisable.

* *Loc. cit.*, Chapter X.

† *Loc. cit.*, p. 202.

Mr. Fowler.

Conditions might be such that neither the under-water nor the box coffer-dam method would seem advisable, and it might then be asked, are there no other plans of construction possible than the phenomenal scheme proposed by Mr. Noble or the monumental one that has been adopted? Although evidently there are many answers possible to such a question, the logical one would seem to be that a movable pneumatic caisson or diving bell might be used to construct the dock in sections, joining them together with tremie concrete, as proposed in the Gayler block design, or by the use of a movable joint coffer-dam, as proposed for the adopted method, and which presumably is similar to that used for the Antwerp Quay Wall.* Although the Noble plan and the adopted plan are undoubtedly sure methods, and for this reason may prove the cheapest, it does not seem to be presumptuous to call attention to the foregoing ideas which may prove worthy of adoption in some future case.

The vital points are that more time should be allowed for the study and design of such large structures, and that more time should be given the bidders in which to study and estimate on the work and submit proposals.

Mr. Taft.

HARRISON S. TAFT,† ESQ. (by letter).—In view of the article entitled "Floating Concrete Caissons" by the writer and Mr. O. D. Jones,‡ Mr. Stanford's paper has been read with the deepest interest. Notwithstanding the varied uses that have been made of floating concrete caissons, the work now being done at Pearl Harbor under the direction of Mr. Stanford's department stands out as one of the most extensive and most scientifically worked out applications of the art, up to the present date; and its successful accomplishment will be another triumph for American engineering. Although a lengthy discussion of the floating concrete caisson problem by the writer would be superfluous, there are several points in this worthy paper on which, it is thought, a few words can well be said.

Mr. Stanford mentions the serious defects found in the concrete first placed under water by tremie operations, and seems to lay special emphasis on the desirability of excluding salt water from within a mass of freshly deposited concrete. As is well known by those most experienced in placing concrete under such conditions, not only does the infiltration of sea water into it cause the formation of the white chalky substance which prevents the proper formation of the plastic concrete into a dense, water-tight, and homogeneous mass, but inexperience in the operation of the tremie leads to results that are practically worthless—that is, in a great many cases. It is of interest to note that, in the construction of a graving dock by Japanese engineers,

* *Loc. cit.*, Fig. 427.

† Seattle, Wash.

‡ *Professional Memoirs*, March–April, 1915, p. 145.

Mr. Taft. special attention was given to the problem of preventing the intermingling of sea water with the green and freshly placed concrete in depositing, under water by the tremie process, a thick layer of concrete as a "seal" for the base of the floor of the dock. The method they used was the reduction of the saline contents of the sea water to a minimum by the introduction into the enclosed coffer-dam, of fresh water from the city water supply, an operation that is said to have met with marked success, and resulted in a first-class concrete sealing job. The coffer-dam was afterward unwatered and the remainder of the dock was completed in the dry with equally marked success. Whether such a mode of procedure would have been possible, or would have eliminated some of the troubles at Pearl Harbor, the writer is not in a position to state.

Mr. Stanford's statement that briquette tests showed that concrete mortar must not be of a less than 1:2 mixture, for a successful use under sea-water conditions, coincides with the results of other experiments by prominent engineers.

The writer notes that Mr. Stanford does not state the chemical composition of the cement being used at Pearl Harbor; it would be of interest if he would furnish some data as to whether he is taking any steps to obtain a special manufactured cement for the work, or is using the daily commercial output of the mill.

The author mentions the rapidity with which the concrete was mixed, namely, batches, of some 1.85 cu. yd. each, at the rate of 40 to 50 per hour, equivalent to a batch every 72 to 90 sec. (approximately 1¼ to 1½ min.). Though it seems to be the aim of some engineers, and especially of contractors, to turn out the greatest possible quantity of concrete per hour, without regard to the time of actual mixing, a great many are beginning to feel, and note in their specifications, that some limit should be put on the length of time a batch is kept turning in the mixer, especially when the concrete is to be exposed to contact with sea water, and still more especially when placed by a tremie. Though a 5-min. mixture may be excessive, it is without doubt far better to mix the concrete as long as 5 min., or even longer, than to shorten the time of the mixing to such an extent as to have the supposedly mixed concrete discharging before the last of the unmixed ingredients have been put into the mixer, a mode of procedure that most surely will cause future troubles, and in a very short time at that, if the concrete is used in structures in contact with sea water. A period of fully 3 min., as called for by some engineers, or a definite number of turns of the machine, such as 25, as used on one Government job of magnitude (all reckoned from the time that the last of the necessary ingredients have entered the mixer), will no doubt give far better concrete than the speed quoted by the author. It is possible that the rate of mixing, as well as the lean concrete, and the inexperi-

ence in using the tremie, account for the poor and worthless results Mr. Taft.
Mr. Stanford says were obtained with the concrete placed in Crib II.

The writer has at times been asked as to the holding-down power of a pile having its head encased in concrete, and as to whether the head would be pulled out of the concrete or the pile pulled out of the ground by the lifting of the concrete structure. One case, at least, which has been cited, was the washing out of a bridge by flood waters, whereby some of the piles were broken off at the under side of the concrete base and others were pulled completely out of the earth and remained embedded in the concrete as when the structure stood erect in its proper position. As well known, the figure usually adopted to determine the carrying power of a pile in compact sands is about 660 lb. per sq. ft. of pile surface, equivalent to about 1 ton per lin. ft. for a 12-in. pile. On the basis of 47 tons upward pull, as noted by Mr. Stanford, and a 5-ft. embedded head, for a 12-in. butt, the calculated resistance is about 3 tons per sq. ft. of contact surface. The resistance, for a pull to destruction of the bond, is apparently unknown.

In giving the results of the experiments with a tremie, Mr. Stanford has added valuable material to a subject on which engineers in actual charge of work have had but few accurate data. In several articles in engineering literature during the past few years, facts of considerable worth have been given to the contracting engineer, as to the best size of tube, etc., and as to operation; but all such data, even that of Mr. Stanford, do not seem to have furnished the engineer with a suitable description of the method of first loading the tremie, in order to prevent absolutely the intermingling of the concrete and the water in the tube. In other words, there is no improvement over the old-fashioned methods of driving the water out of the tube during the loading period, and at the same time keeping the initial loading of the tube and its going into operation under absolute control, irrespective of the depth of the water or the size of the tube.

In depositing the subaqueous concrete around the tubes of the Detroit River Tunnel, "a wadding of cement sacks was at first placed in each tremie on top of the water to prevent the concrete from dropping through while filling the pipe".* Batches of concrete were then poured in on top of the wadding, the slight raising of the tremie permitting part of the water to run out and the stopper to run part way down the tube. More concrete was then poured in, etc., until the stopper finally passed out of the bottom of the tube and the outflowing concrete formed a seal around it. After some tests it was found that dry concrete could be used to take the place of the sack wadding, and such a mode of procedure was adopted after the work was well under way.

* *Engineering News*, March 17th, 1910, p. 320.

Mr. Taft.

In place of a sack wadding as a stopper, some engineers have used paper bags filled with dry concrete; others have used a wadding made by dumping in a pail of sawdust on top of the water before pouring the concrete into the hopper of the tremie. Mr. G. W. Rear has devised a mode of using a stopper consisting of some 7 or 8 in. of planer shavings covered with 1 in. of cement, the buoyancy of the shavings and the resistance of the column of water below preventing the stopper from falling too rapidly down the tube, the concrete itself acting as the seal. As stated by Mr. Stanford, at Pearl Harbor, a bag filled with straw was inserted and descended in the tube when the concrete was placed above it.

Although the foregoing and other methods of loading a tremie usually call for the assembly of a large quantity of concrete for the initial pouring, they also leave the operation practically uncontrolled by the men in charge, and the discharge of some tons of weight into the tremie at one operation is apt to set up considerable vertical movement, especially when the tremie is hung from the end of a boom on a scow, thus interfering seriously with successful loading.

The introduction of such matter as sawdust, shavings, straw, paper bags, etc., may be of absolutely no importance in the placing of subaqueous mass concrete, yet in cases of reinforced concrete columns, from 3 to 4 ft. in diameter, as used at San Francisco and in the Puget Sound Navy Yard, the introduction of such foreign matter may affect seriously the permanency of the structure. The elimination of all foreign matter from concrete placed in sea water should be absolutely insisted on, if the best results are to be guaranteed. It was in connection with just such a guaranty that the writer was led to devise an all-concrete stopper, mechanically controlled in its descent through the tube, even though the lower end of the latter was far above the mud line and freely swinging in the water. A brief description of this stopper, it is thought, will be of interest.

As shown in Fig. 20, the stopper is of concrete, having a circular disk for its base and a frustum of a cone for its head, the two parts being made monolithically. It is built in two parts, with a hole at the center, so as to permit them to be placed around a bolt and rest on a washer at the pointed end. The diameter of the stopper should be such as to suit the tube, with sufficient clearance to permit of a free descent. The bolt is suspended by a wire or rope running to some kind of controlling device, such as a winch, to provide for lowering the stopper through the tube. If desired, a plastic washer of soft leather or rubber may be placed at the top of the stopper, as shown. The two parts of the stopper are first placed around the bolt in the upper part of the tube and lowered slightly. Concrete is then poured in above, and the stopper is slowly lowered as more concrete is poured in above it. When the stopper has reached the lower part of

the tube and is ready to pass out, the tremie having been raised sufficiently above the ground, the unbalanced couple formed by the supporting bolt and the weight of the concrete above the stopper causes the two halves to fall apart and permit the outflow of the concrete to form a seal about the bottom of the tube, the hopper having been fully loaded in order to prevent any interruption in the column of concrete in the tube after the opening of the stopper. The bolt is then withdrawn and kept for the next loading. As the two parts of the stopper remaining in the mass are of concrete, there is no foreign matter in the deposited concrete, as is the case with some of the older modes Mr. Taft.

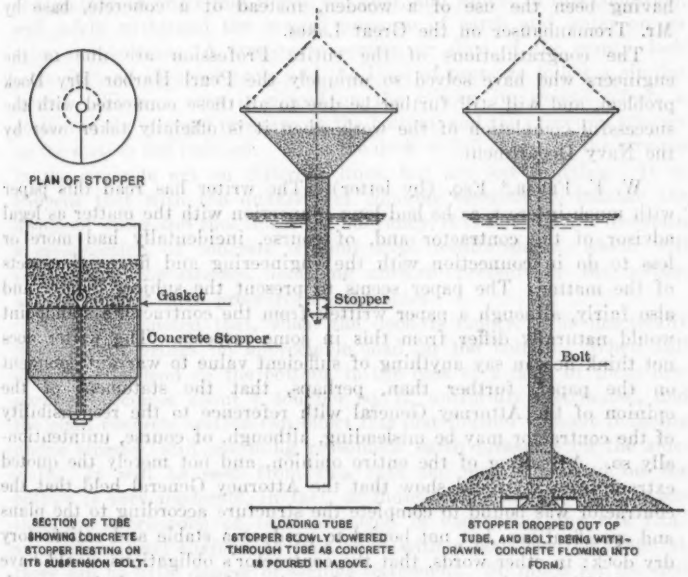


FIG. 20.

of operation. Though the writer has not yet had an opportunity for an extensive trial of this device, it appears evident that it provides an absolute means of preventing any intermingling of the water and the concrete, and at the same time there is positive control of the whole operation.

Inasmuch as the prime factor in securing a successful use of concrete under sea-water conditions is a pre-moulded, air-cured structure, as pointed out by the writer,* there can be no doubt that the use of structures of such a type by engineers is bound to become more

* In "Floating Concrete Caissons."

Mr. Taft. general, as the years go by, the successful culmination of the Pearl Harbor project, no doubt, doing much to encourage the engineers of America in a still further and more extensive use of such a mode of construction and in a more liberal utilization of concrete in harbor development and sea structures. Though it is a matter of record that the first floating concrete caisson was built without any reinforcement whatsoever, all subsequent ones have been on a typical reinforced basis. The extensive use of structural steel, as adopted for the Pearl Harbor Dry Dock, apparently is the first in such a structure, thus marking the fourth step in the development of the art, the third having been the use of a wooden, instead of a concrete, base by Mr. Tromanhauser on the Great Lakes.

The congratulations of the entire Profession are due to the engineers who have solved so uniquely the Pearl Harbor Dry Dock problem, and will still further be due to all those connected with the successful completion of the work when it is officially taken over by the Navy Department.

Mr. Frear. W. F. FREAR,* ESQ. (by letter).—The writer has read this paper with much interest, as he had long connection with the matter as legal advisor of the contractor and, of course, incidentally had more or less to do in connection with the engineering and financial aspects of the matter. The paper seems to present the subject lucidly and also fairly, although a paper written from the contractor's standpoint would naturally differ from this in some respects. The writer does not think he can say anything of sufficient value to warrant comment on the paper, further than, perhaps, that the statement of the opinion of the Attorney General with reference to the responsibility of the contractor may be misleading, although, of course, unintentionally so. A reading of the entire opinion, and not merely the quoted extract from it, would show that the Attorney General held that the contractor was bound to complete the structure according to the plans and specifications, but not bound to produce a stable and satisfactory dry dock; in other words, that the contractor's obligations would have been fulfilled if the structure had been brought to completion according to the plans and specifications, even though it should collapse afterward.

It was a matter of gratification, both to the Government and the contractor, to be able to arrange for the completion of the work on the new plan under a supplemental agreement, and it is to be hoped that in due time a dock satisfactory to all parties will result.

Mr. Reed. P. L. REED,† M. AM. SOC. C. E. (by letter).—Supplementing the detailed account of the Pearl Harbor Dry Dock given by the author,

* Honolulu, Hawaii.

† Charleston, S. C.

as regards location, geology, history, finances, design, and construction so far as it has progressed, a few general features may be emphasized. Mr Reed.

There have been two primary problems in the design of this structure: First, to design a dry dock which would safely dock ships of specified maximum dimensions, and would withstand indefinitely the stresses brought on it in this service; second, to design a dry dock which could be safely constructed on the Pearl Harbor site.

In this case, the first problem was much the simpler. Given the interior clearances, it is required to design a concrete box which will weigh when empty approximately as much as the water it displaces at high tide, in order that it will not float, and which, in this condition, will safely withstand the inward pressure of earth and water on its sides and bottom. A transverse section or slice of the empty dock is, in fact, an inverted arch, and the lines of pressure follow a similar course. When a ship is placed on the blocks, its weight offsets a portion of the upward pressure on the bottom, and the principal stresses on the section are reduced. When the dock is full of water, the stresses in the concrete act on different lines, but are less exacting. It so happens that with the quantity of concrete required to balance the displaced water and thus offset the buoyancy, it is not difficult to obtain a cross-section of concrete without steel reinforcement, which will safely carry the stresses, assuming uniform pressure on the bottom and reasonable earth pressures on the sides.

It will be noted that, when the dock is empty, it is practically floating and brings no appreciable load on the bottom. When full of water, the load on the bottom, in the case of a dock of these proportions, becomes about 1 ton per sq. ft. uniformly distributed, and the plain concrete section can effect this distribution without inducing undue stresses, still assuming reasonable earth pressures on the sidewalls. It will be seen from this that the maximum load on the bottom is very moderate, in fact, as indicated in the paper, it is less than the overburden of earth removed in excavating the site from an original level surface, so that, even with a poor bearing material, there can be little danger of serious settlement. So much for the first problem.

The second and much harder problem was to design such a concrete box that it could be positively and safely constructed in a freely water-bearing soil where a coffer-dam was found unavailing, as the water came up through the bottom. As the empty concrete box weighs as much as the water displaced when it is in place on the bottom, it could not be constructed in whole or in complete sections and floated to place, as has been done with some concrete caissons and sea walls, and it is very undesirable to deposit concrete under sea water where it can be avoided. What has been done in the adopted design, as explained in detail in the paper, is to provide a portable floating steel

Mr. Reed.

coffer-dam by which transverse sections or slices of the dry dock can be partly constructed on a floating dock, floated off, sunk in place, and held down by a tank of water while the sections are being completed in the dry. This method of construction brings in temporary stresses which are greater than those which the finished dock must sustain, and a large quantity of steel framing and reinforcement is required, which remains in the finished dock and increases its strength correspondingly, but which would not be necessary, except for the unusual method of construction required.

The steel coffer-dam combined with and surrounding the steel box which serves alternately as a water tank to hold down the uncompleted, and therefore buoyant, sections of the dock, and as a float to lift the coffer-dam as a whole from a completed section and place it on a new dock section partly constructed in the floating dock, is the ingenious invention of the author to meet the unusual conditions described, and constitutes an effective and novel combination of construction appliances.

Mr. Harris.

FREDERIC R. HARRIS,* M. AM. SOC. C. E. (by letter).—The writer is much interested in this paper on the Pearl Harbor dry dock, giving as it does a description of the geology of the location, the history of the design, the difficulties encountered in the construction, and the evolution of the plan on which the work is now being done.

In the latter part of March, 1913, shortly after the disastrous collapse of February 17th, the writer, under orders from the Secretary of the Navy, visited Pearl Harbor with Mr. Stanford, and spent 11 weeks investigating and examining the work, and inquiring into the causes of failure. Mr. Stanford was then, as he is now, Chief of the Bureau of Yards and Docks, and had many duties and responsibilities other than this particular work, so that, during the 2 weeks that he remained on the Island, much of the planning for a systematic investigation, together with the charge of the actual work thereof, was directly in the writer's hands. After Mr. Stanford's departure, in addition to this work, the writer, with E. R. Gayler, M. Am. Soc. C. E., the officer in charge, and Mr. Gordon, was a member of a Board to investigate the foundation conditions of the site, and this work was carried on independently of the principal investigation, although many of the data obtained for the latter were used in connection with the work of the Board.

Geological.—In addition to the comment in the paper on the geological formation, the writer would like to bring out the impression, made by conditions on the Island of Oahu and other islands of the Hawaiian group, on an engineer familiar with work on the main con-

* Philadelphia, Pa.

tinent, where his experience would furnish little or no precedent to guide him in an understanding of the Hawaiian soil and foundation conditions. As an illustration, ordinarily, in the older main-continent formation, when the over-burden of earth and soils has been removed and rock is encountered, and after proper precaution has been taken to be sure that it is not a boulder, the assumption is well warranted that it is ledge rock and extends indefinitely downward. In the Hawaiian Islands, frequently a volcanic rock is encountered, and soils or disintegrated rock are found below it. Geologically, the islands are very recent, and earth or soil similar in character to the soils of the continent would yield and compact considerably under a foundation pressure, which, in the United States, would be considered thoroughly safe, a probable and logical explanation being that such surface soils, or soils comparatively near the surface, have never been subjected to the great pressures of an over-burden, later moved by erosive and weathering action. They are so recent geologically that there has been no opportunity for the working over with mechanical and chemical changes that will occur with the passing of future ages. Mr. Harris.

Borings.—On page 257, and in what follows, Mr. Stanford has mentioned the exploration borings, made under the direction of C. W. Parks, M. Am. Soc. C. E., in 1908, and the check borings, which were made under the writer's directions. In further explanation of this information, the writer, for a long time, in common with many other engineers, has believed that it is frequently difficult fully to appreciate and understand subsoil conditions and formations from the information obtainable and available by the wash-boring method of exploration, especially when the investigation attempts to bring out the compactness and imperviousness of the soils in place, the difficulty being emphasized by the recent volcanic formation of these islands and the extreme uncertainty of properly classifying materials brought up by wash-borings where experience and judgment would be predicated on continental formations.

Previous to sailing for the Islands, the writer had a conference with A. L. Parsons, M. Am. Soc. C. E., on this matter, and, on arriving, received from him some advertisements and descriptive matter relative to the Calyx double-core barrel drill for soft ground exploration. An attempt was made to secure such a drill, but the nearest one was then in El Paso, Tex., and to obtain it would have involved considerable time and delay. A smaller rotary drill of this type was in use by the army engineer in Honolulu, but it was not furnished with a double-core barrel, so that it was attempted to secure solid cores by the driven-pipe method, as described by Mr. Stanford on page 258. The writer was satisfied that the results being obtained closely represented the subsoil conditions, but the earlier reports from Mr. Gayler, and

Mr. Harris. the opinion of the contractor's engineer, Francis M. Smith, M. Am. Soc. C. E., appeared to be that the soils underlying the work were of a semi-liquid or plastic character, and that the failure of the work had been caused by liquid mud pressure, that is, the pressure of the material composing the banks on the lower so-called lava mud. Figs. 8 and 9 are typical of the cores obtained from the driven pipe. A great many cores and samples obtained in the boring operations show no evidence of plasticity or a liquid condition, but Mr. Smith believed that possibly the driving of the core barrel ahead of the casing might have compacted the material, especially the so-called lava mud, and driven out the water, so that although the obtaining of cores was proceeded with by the driven-pipe method, arrangements were made to use the rotary Calyx drill. Four borings on the bank were made by this drill, the object being to check up by this method of boring, which did not involve compacting the cores. The cores were inspected as they were secured, and their specific gravity or weight submerged in sea water was taken, the object of which will be made clear hereinafter.

In addition to obtaining the solid cores and their weight in sea water, it was decided to investigate the porosity or perviousness of the subsoil, especially to locate, if possible, continuous impervious strata by permeability tests. In some recent core borings in ledge rock, the perviousness or leakage has been ascertained with two plungers or pistons, closely fitting the bore, by supplying water under pressure to the space between them. As the operations were carried on at various elevations, it was, of course, impossible to apply this method, and the following expedient was adopted: Directly after withdrawing the driven-pipe core barrel, the casing pipe was filled with water to a height of several feet above tide level, thus subjecting the subsoil at the foot of the casing pipe to a pressure or head of several feet of water, and then the loss of water in the casing pipe was observed for a time. As the joints in the casing pipe were not leaded, undoubtedly, in many instances, slight leakage occurred, so that very small outflows of $\frac{1}{8}$ in. per min. or less were considered as representing practically an impervious soil. These observations were continued and recorded. The result was a strong indication that the so-called lava mud and this same material mixed with some coral was impervious and continuous over the entire area, forming a complete water-tight diaphragm. The coral materials above this lava formation were somewhat pervious, and those below it were more pervious, and, in instances, quite freely water-bearing. The indications in all these observations were that the water present in the soils was under a ground-water or tide-water head equivalent to about that of tide level. It is stated by Mr. Stanford that "the character of the mud stratum has been of vital importance throughout

this work." Although, geologically, it might be termed "mud", its physical characteristics would be defined more clearly to an engineer by the designation "clay". Under the microscope, it was more granular, the contained water was hygroscopic, and it was deficient in aluminum, as compared with continental clays. In making and burning briquettes, in some instances they were partly vitrified, and in others powdered and disintegrated. Samples immersed in water did not in all instances show the characteristics of continental clay, nevertheless, they did not dissolve readily and break up, but preserved their shape to such an extent that it was possible to immerse them in water and obtain their specific gravity and still retain them as effective samples. Plate X, made up from the 1913 borings, indicates the probable strati- Mr. Harris.

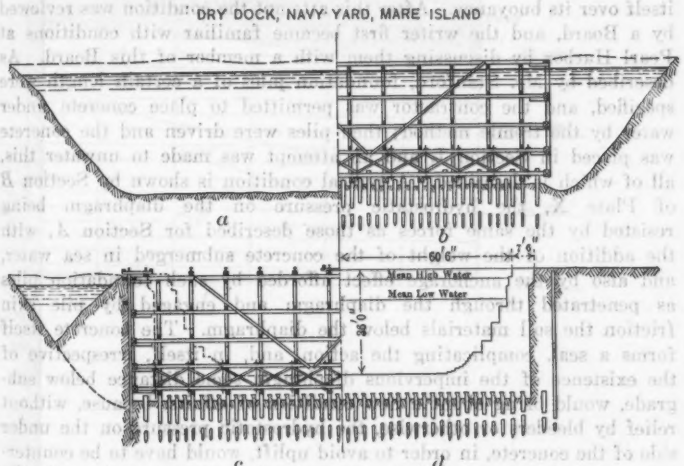


FIG. 21.

fication on the site of the structure. It is possible, of course, that the so-called lava mud stratum was not continuous and complete, but the borings appear to indicate that it was. The cross-hatched, shaded portion of the stratification is the apparently impervious stratum or diaphragm. It will be noted that at a considerably lower elevation, beyond the depth of direct interest in the investigation of this work, a somewhat similar stratum of great depth is found.

Design and Construction.—Mr. Smith, in charge of the work for the San Francisco Bridge Company, had successfully completed the dry dock at Mare Island, Cal., previous to taking charge of this work, the method of construction being shown in Fig. 21, in four stages, which are largely self-explanatory. The work was completed by

Mr. Harris.

making the excavation, building large timber cribs, sinking these, driving around the periphery heavy timber sheet-piling, backing this up with the earth back-fill, and then pumping out the interior. A similar method was adopted for this dock, it being planned to use five sections or cribs, as explained by Mr. Stanford.

Section *A* of Plate X is typical of subsoil conditions in Section I, and shows the operations there in the first attempt to unwater it, called Failure No. 1. The rising of the crib can be explained by the removal of the weight of water from its interior—the hydrostatic pressure on the under side of the diaphragm being resisted by the weight of the water left in the interior of the crib, the weight of the saturated soil above the diaphragm, and the excess weight of the crib itself over its buoyancy. After this attempt the condition was reviewed by a Board, and the writer first became familiar with conditions at Pearl Harbor by discussing them with a member of this Board. As described by Mr. Stanford, foundation piles of a certain length were specified, and the contractor was permitted to place concrete under water by the tremie method; then piles were driven and the concrete was placed in Section I, and an attempt was made to unwater this, all of which is described. A typical condition is shown by Section *B* of Plate X, the hydrostatic pressure on the diaphragm being resisted by the same forces as those described for Section *A*, with the addition of the weight of the concrete submerged in sea water, and also by the anchorage effect afforded by such foundation piles as penetrated through the diaphragm and engaged by pile skin friction the soil materials below the diaphragm. The concrete itself forms a seal, complicating the action, and, in itself, irrespective of the existence of the impervious diaphragm some distance below subgrade, would bring about a new condition of affairs, because, without relief by bleeders or otherwise, the hydrostatic pressure on the under side of the concrete, in order to avoid uplift, would have to be counterbalanced by the weight of the water in the interior of the crib, the excess weight of the crib, the submerged weight of the concrete seal, and the anchorage effect of the foundation piling, in turn dependent on the submerged weight of the mass or prism of earth engaged by these foundation piles. As given by Mr. Stanford on page 241, the average length of the 1 966 piles driven in Section I was 23.78 ft., and although the resistance to up-pull for a single pile is dependent on the skin friction of the pile, or the actual mechanical anchorage in the case of piles with enlarged bases, in either case it is limited by the weight submerged in water of the conical but irregular masses of earth surrounding the pile that would have to be lifted; the anchorage effect in a group of piles driven closely together would be less than the sum of the individual effects of a like number of single piles, and in turn would be limited by the irregular prism or mass of

PLATE X.
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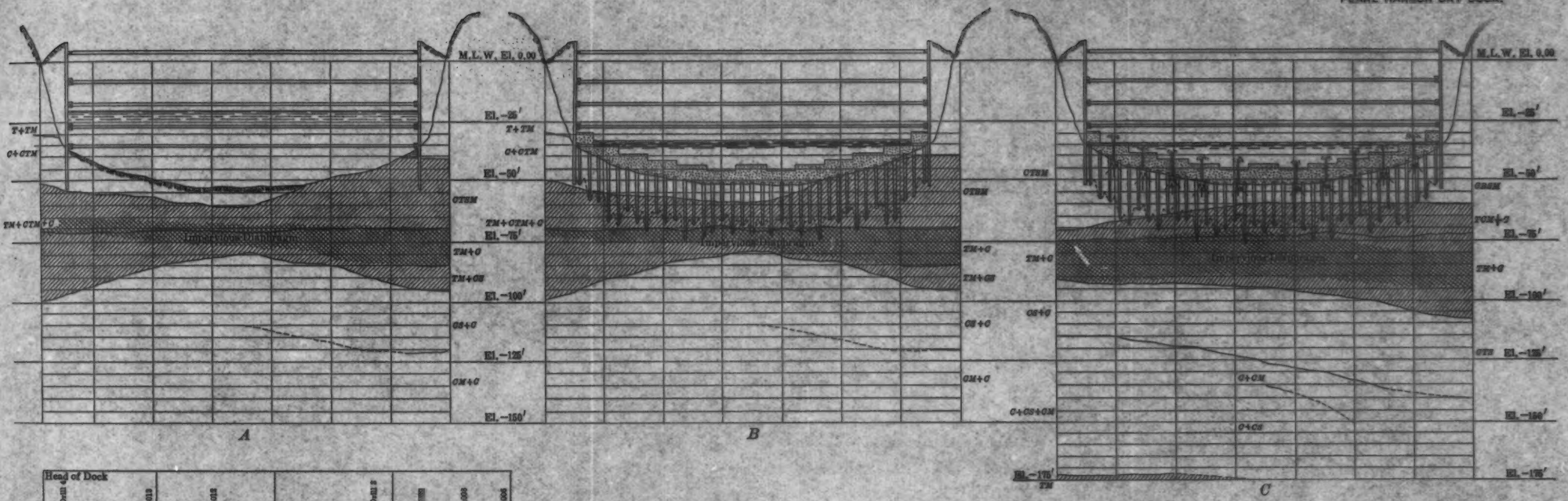
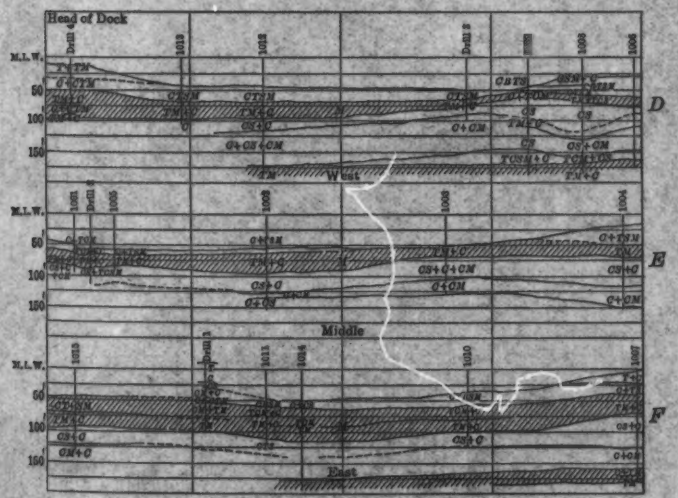
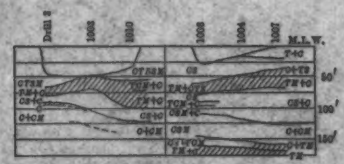
HARRISON
PEARL HARBOR DRY DOCK.





LONGITUDINAL SECTIONS



TRANSVERSE SECTIONS
H G

STRATIFICATION INDICATED TAKEN FROM REPORT OF PROFESSOR T. A. JAGGAR JR., GEOLOGIST

C=Coral
T=Tuff
B=Basalt
S=Sand and Gravel
M=Lava Mud or Clay

PEARL HARBOR DRY DOCK

DIAGRAM SHOWING SUBSOIL CONDITIONS AND TYPICAL SECTION IN CRIBS I AND II FOR 1st, 2d AND 3d FAILURES

earth pinned together or engaged by pile skin friction, so that the exact effect of the anchorage piles would depend not so much on the average length of the piles as on the relation of the long piles to the average. Mr. Harris.

In making a check computation on Failure No. 2, the rising of the crib was noticeable with water removed from the interior to from 32.8 to 34.3 ft., and this would give the mean depth or thickness of the mass of earth lifted as 22 ft., as compared with an average length of piling of 23.78 ft.

The next failure, or rather the final collapse, occurred on February 17th, 1913, in Section II, the typical conditions of which are shown by Section *C*, Plate X. As stated by Mr. Stanford, the average length of piles in this section was 20.53 ft., giving by similar computation the mean depth of the mass of earth lifted as 18.5 ft.

Referring to the sixty 2½-in. vertical drain-pipe bleeders described by Mr. Stanford, on examination of Section *C*, Plate X, it will be readily seen that these merely served to drain the confined soil above the so-called lava mud diaphragm, and did not help in any way to relieve the pressure under the diaphragm itself, and to that extent their effect toward averting failure was negligible, if of any use at all. The existence of the bleeders would serve to relieve more or less, and decrease, the hydrostatic pressure on the under side of the concrete seal, but with the existence of an impervious stratum or diaphragm some distance below sub-grade, by draining out the ground-water in the coral above the so-called lava mud, they might decrease the total weight of ground-water above the diaphragm, and to that extent decrease the resistance to uplift on account of the pressure below the diaphragm, and in this way possibly bring about an actual upward movement earlier than would be the case if these bleeders had not been put in. It is extremely doubtful whether any such conditions existed, but it is readily seen that the value of the bleeders was negligible, as they affected the hydrostatic pressure exerted on the under side of the so-called lava mud stratum, which was practically impervious.

In the computations referred to, 54 lb. was taken as the weight of the soil immersed in sea water; an average of actual observations gave 60 lb., but this was reduced by 10% for possible compression in securing samples by the driven-pipe method.

Suggested Plans.—In June, 1911, after the failure to unwater Crib I, and before the change was made specifying foundation piles, with permission to place a certain quantity of concrete under water by the tremie method, the writer became familiar with Pearl Harbor dry dock conditions, and on June 11th, in correspondence with P. L. Reed, M. Am. Soc. C. E., a member of the Board, suggested

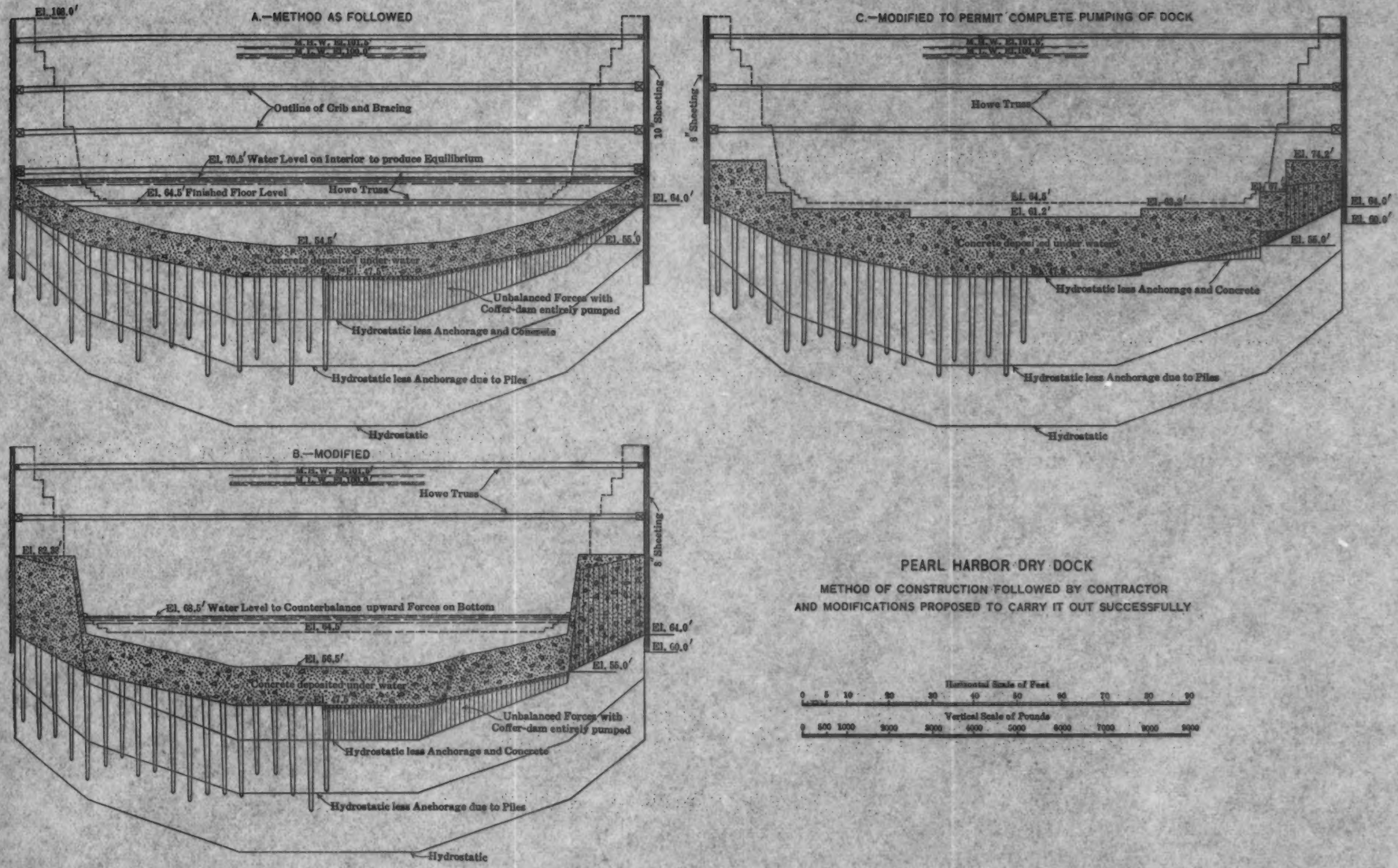
Mr. Harris.

that, if a tie were considered necessary for anchorage purposes, especially to provide for the construction condition, rectangular caissons be used instead of piles, in a manner somewhat similar to what was then being done on Dry Dock No. 4 at the New York Navy Yard, of the construction of which the writer was in charge, and called attention to the necessity, during this construction stage, of engaging by anchorage and a stratum or prism of earth of a mean depth of 35 ft., in addition to the submerged weight of an 8-ft. layer of concrete.

As stated by Mr. Stanford, the view was taken that the anchorage condition during the construction stages was a problem for the contractor, and it was doubtful whether any anchorage of foundation piles would be required for the completed dock. The Government, however, did provide for paying for foundation piles with an average penetration of 35 ft., with an arrangement for a price per linear foot, so that payment would only be made for lengths actually driven. The writer can state without hesitation that it was impossible to drive piles to a much greater average depth than was secured, which, in the case of Section I, was slightly in excess of 20 ft., and in Section II slightly less than 24 ft., so that perhaps the first suggestion for a change in plan was made by the writer when he proposed the substitution of pneumatic caisson piers and anchors. However, with the information available at that time, there was then apparently no serious reason for considering or adopting a costly change in plan.

In April, 1913, shortly after arriving in the Hawaiian Islands, and after becoming familiar with the conditions, the writer was strongly of the opinion that two general courses were available for the completion of this dry dock: First, following the lines up to that time proceeded with, by the construction in an open coffer-dam, holding down the structure during construction against uplift and flotation by added anchorage or weight, and, as such added anchorage could not be secured by driving deeper anchor or foundation piles, by either placing considerably more concrete by the tremie method under water before unwatering, or by adding weight in the form of rock ballast. The quantity of rock ballast that would have been required in one section was considerable, in fact, this plan seemed impracticable. The second plan was to adopt the familiar method often used in bridge pier construction, where the unwatering of the site and the building of an open coffer-dam is impracticable or undesirable; that is, the preparation of a level foundation and the sinking of the structure on this prepared foundation through the water without unwatering, that is, the use of floating coffer-dams or caissons. In February, 1910, while the writer was in charge of the dry dock construction at New York, he had suggested to Mr. Frederick Holbrook, of the Holbrook, Cabot and Rollins Corporation, the placing of the entrance of that dry dock by using a similar caisson, and had drawn up a tentative

PLATE XI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXX, No. 1354.
HARRIS ON
PEARL HARBOR DRY DOCK.



PEARL HARBOR DRY DOCK

METHOD OF CONSTRUCTION FOLLOWED BY CONTRACTOR
AND MODIFICATIONS PROPOSED TO CARRY IT OUT SUCCESSFULLY

plan for the work on these lines for the purpose of avoiding the construction of the usual open coffer-dam at the entrance. The plan was discarded as more expensive than that finally used. Further, in September, 1912, as an associate with William Barclay Parsons, M. Am. Soc. C. E., on a project for dry dock construction in South Brooklyn, the writer made studies, outline design, and estimates for a dry dock, using coffer-dams or caissons, building the dock in sections, and constructing these sections in one of the timber graving docks available at the Erie Basin plant. The writer had with him during his visit to Pearl Harbor one of these sectional sketches, and discussed the matter with Mr. Stanford, who then believed that it was not advisable or necessary to consider so elaborate a plan. He also discussed this plan with S. H. Hindes, M. Am. Soc. C. E., President of the contracting firm, and Mr. Smith. It is noted that Mr. Stanford, on page 269, states that the writer submitted two plans in a letter dated September 26th, 1913. It is believed that this is unintentionally misleading, as the writer, on his return from the Hawaiian Islands, at the end of June, 1913, in a report to the Secretary of the Navy, dated July 1st, 1913, and written some time before this in the Hawaiian Islands, states: Mr. Harris.

"In view of what has been developed on this work, it is quite probable that some method of construction dependent upon the use of either timber or reinforced concrete floating caissons would lend itself more readily to the construction of the dry dock on this site. I have in mind the possible dividing of the dock into ten sections, each caisson to be the full width of the dock and 100 feet long; concrete masonry work of the dock proper to be built in the dry in the interior of these boxes, the added weight assisted by weight of flooding water to sink these boxes or caissons to place upon the prepared pile foundation. In Sections 1, 3, 4, and 5, working chambers could be provided to be operated under air pressure, these chambers later filled with concrete surrounding the heads of piles already driven; in Section 2 all of the concrete masonry to be placed in the open, no working chamber or air pressure to be used, and the entire mass sunk in place as a gravity structure resting upon the prepared pile foundation, or the same method outlined for Section 2 to be used for Sections 1, 3, 4, and 5. This method of construction would permit of reinforcing with steel rods the floor or bottom of the dock so as to insure practically equal distribution of load over the dock bottom."

In the summary or letter of transmission, of July 1st, 1913, this statement was contained:

"With foresight and care it should have been possible, and still is possible, to employ the construction methods successfully that were employed previous to the failure, although certain modifications of the details of the design may be required to insure success of this method; such, for instance, as the placing of more concrete in the

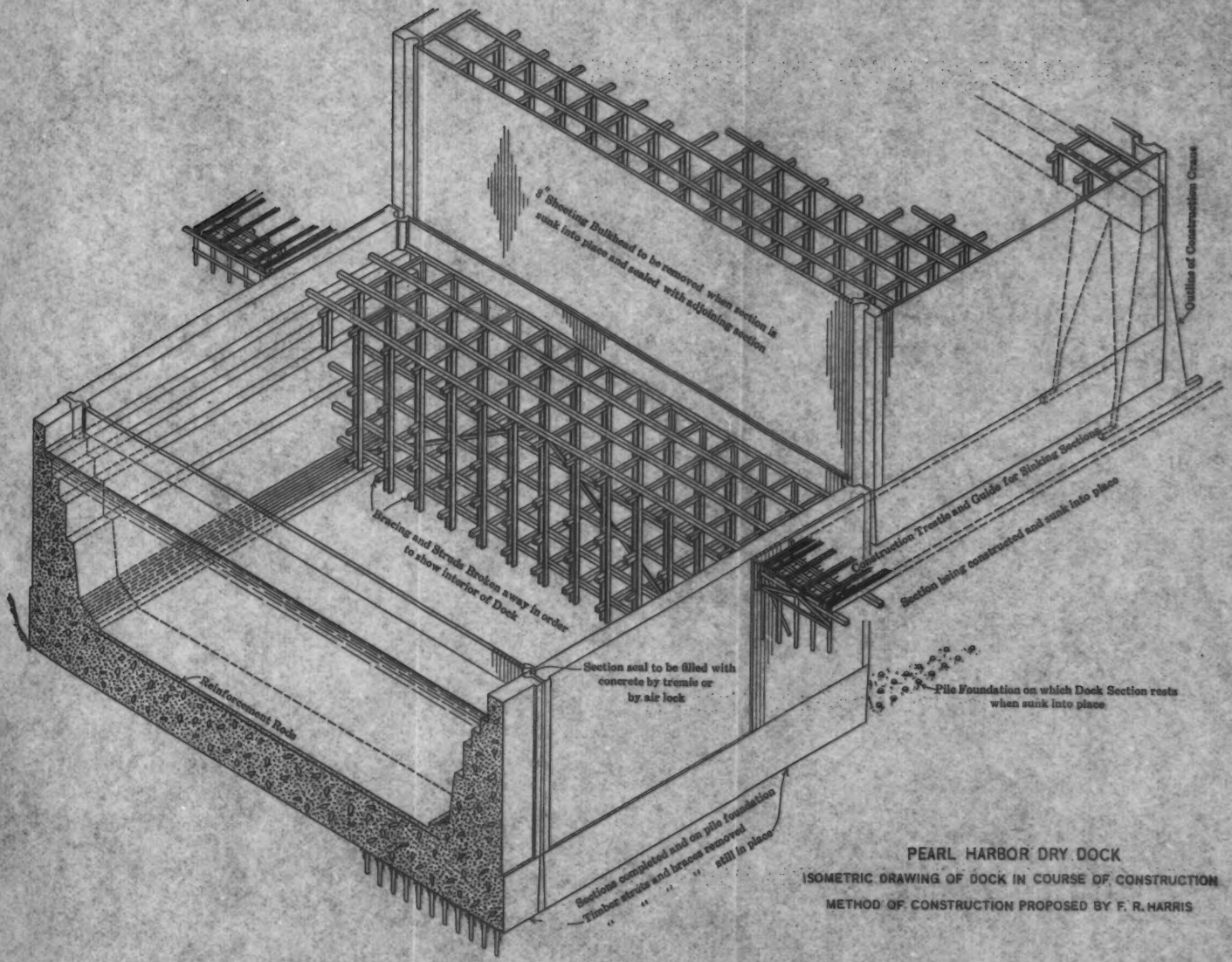
Mr. Harris. bottom, and the application of movable weight to the bottom, all sufficient in amount to restore equilibrium during construction."

Immediately on the writer's return from the Hawaiian Islands, he started the development of the plan of the sectional floating caisson, recommendation of which was made in his report of July 1st, and on its completion it was forwarded on September 26th, 1913, the date referred to by Mr. Stanford. Only one plan was submitted—that of building the dry dock by sectional floating caissons—this plan, as may be seen by reference to Plates XII and XIII, being practically a sketch or pictured drawing. The details were not worked out, nor were any complete computations made. The other matter referred to as a second plan by Mr. Stanford was merely a graphical demonstration of the method used by the contractor, the probable cause of failure by unbalanced forces, and a suggestion as to how equilibrium could be brought about by the addition of under-water concrete, all shown by Plate XI. The adoption of the floating caisson plan was urged in this report as more certain and expeditious, and as a plan that, in view of the conditions developed by the failure of February 17th, 1913, would remove considerable of the danger and hazard. The time from July 1st to September 26th was taken up in preparing the picture sketches and consulting with several engineers and contractors of experience, in order to obtain from them their views as to the expense of this type of construction. The writer did not then hold the view, nor does he now, that anything so very new or remarkable was being recommended, but stated in his report:

"It is believed that this is an untried method for the construction of dry docks, but a method that has been frequently used for the placing of bridge piers, although probably not on so large a scale nor involving the placing of so many separate sections to be later connected and sealed."

Up to October, 1913, the writer had not been informed of the employment of the late Alfred Noble, Past-President, Am. Soc. C. E., by the Government, or that he had visited the Hawaiian Islands. Shortly after his return, early in October, 1913, Mr. Noble communicated with the writer and informed him of his visit, and that he was making some studies in the matter. The writer, at his request, forwarded to him the studies contained in the supplementary report of September 26th, Mr. Noble informing him that he already had his report of July 1st and had taken that out to Pearl Harbor with him. Mr. Noble never claimed that his recommendation, that the sectional floating caisson plan of construction be used at Pearl Harbor, was other than an elaboration and development of the recommendation of the writer, for how could he, as he took the writer's report of July 1st, containing the suggestion of this method, to Pearl Harbor

PLATE XII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXX, No. 1354.
HARRIS ON
PEARL HARBOR DRY DOCK.



PEARL HARBOR DRY DOCK

ISOMETRIC DRAWING OF DOCK IN COURSE OF CONSTRUCTION

METHOD OF CONSTRUCTION PROPOSED BY F. R. HARRIS

with him? In order to correct any erroneous impression, the writer would state that the information given, that Leonard M. Cox, M. Am. Soc. C. E., had in 1911 proposed the adoption of this method of construction, is news to him, as previous to his visit to Pearl Harbor and to submitting either the report of July 1st or the supplemental report of September 26th he had not found any such suggested plan among the papers and data given to him in connection with the Pearl Harbor investigation, the complete records of which were supposed to have been placed at his disposal, nor did he receive any intimation that such a plan had been proposed or existed. Mr. Harris.

The construction of bridge piers by this method is not new; it was applied successfully in breakwater construction in Zeebrugge, Belgium, in 1905; in Barcelona, Spain, in 1906; in Touapse, on the Black Sea; in Talcahuana, Chili, in 1908; in Bilbao and Bizerta; and in the United States under patent secured by W. V. Judson, M. Am. Soc. C. E., in Algoma, Wis., etc.; and on quay wall construction in Rotterdam; Norresundby, Denmark; Passau, on the Danube; and on a large and extensive scale in connection with pneumatic pressure and diving-bell work at Antwerp, Belgium. A dry dock built at Toulon, France, was constructed in a large steel caisson with a working chamber beneath; an extension to the dry dock at Livourne, France, by a single pneumatic caisson; and the dry dock at New York Navy Yard, constructed under the writer's supervision and design, by surrounding the site with a cut-off wall sunk into place in pneumatic caissons with central piers or anchors placed by the same method. The writer believed that the method recommended by him in the report of July 1st, 1913, was new and untried for dry dock construction, and on this assumption filed an application with the United States Patent Office on October 16th, 1913, and a patent was granted to him on April 6th, 1915. In making preliminary application for a patent, in August, 1913, the writer went on record with the Patent Office to the effect that the Government was to be granted free use of the patent for dry dock construction.

After the submission of the report referred to, the writer had no further connection with Pearl Harbor dry dock matters until January, 1914, when he was consulted relative to taking charge of the completion of the construction of this dry dock, on the same plan and construction methods being followed out previous to the failure. In his opinion this gave little promise of a successful completion of the dock, and involved great hazard and uncertainty. He then for the first time had the opportunity of examining the report and recommendations made by Mr. Noble, and was much gratified and reassured to ascertain therefrom that such a careful and conservative engineer held the same views, and had recommended the adoption of the sectional floating caisson construction for the dry dock. After con-

Mr. Harris.

siderable negotiation and conference. Mr. Cox and the writer were designated to prepare the necessary new studies and plans for the construction of the work. The writer devoted himself to the development of a suitable sectional floating caisson design, Mr. Cox confining his attention to some means of adhering to the original plan by using, for holding-down purposes at the sides, heavy concrete blocks, and at the center a steel ballast tank, very similar in general proportions and dimensions to the ballast tanks of the interior of the coffer-dam boat shown in Figs. 13 and 14.

Mr. Cox did not at that time inform the writer of having suggested or recommended the adoption of the sectional floating caisson method of construction, and in fact at that time was opposed to the adoption of this plan, although the writer has been more recently informed that at some time he discussed with the contractor's engineer, Mr. Smith, and made a rough sketch of a reinforced concrete floating dry dock that might be sunk as a unit and filled with concrete.

Mr. Stanford states on page 269 that the plan recommended by the writer was to construct the base or hull for each unit on launching ways. The writer believed that this would be the most economical and expeditious method of construction, but did not confine his recommendations within such narrow limits. In fact, in his applicaton for the patent, which was granted, he states:

"The floating box or caisson (*a*) shown in Fig. 1 may be constructed in the dry on inclined ways and then launched into the water, or it may be built in a shallow, excavated basin, this being then flooded, or by any other method whereby the floating caisson can be built in the dry and subsequently floated in the water to its proper position. Each box is then to be surmounted by a timber, concrete, or steel extension or lagging * * *."

The writer trusts that he may be pardoned for reference in this discussion to the patent granted to him. It was not taken out with any mercenary motive, nor with a view of any monetary return. He has honestly been under the impression, in so far as the adopted design used in the construction of this dry dock is concerned, that the recommendation for the use of the sectional floating caisson method of construction came from him. It would appear that it certainly first came from him after the failure of February 17th, 1913, when Mr. Stanford and the writer were instructed to investigate the failure of this work and recommend a means of overcoming such difficulties as might be found to exist, and when there was occasion to recommend a radical departure in the design. The writer believes that considerable remains to be done in carrying out the projected construction, from which by no means all the hazard and uncertainty have been removed. The construction of these large floating sections, the placing

PLATE XIII.
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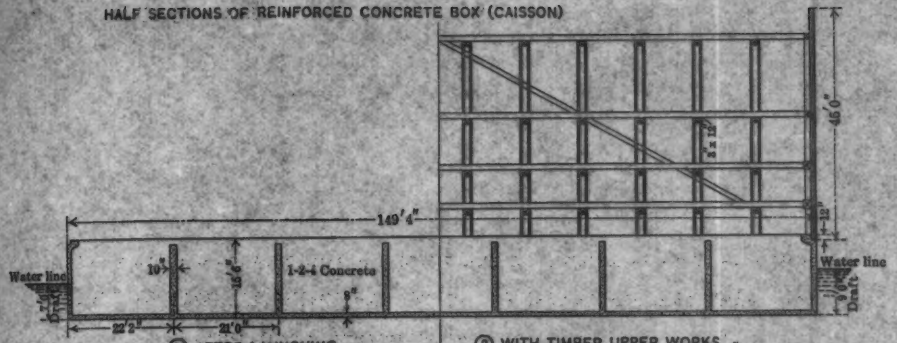
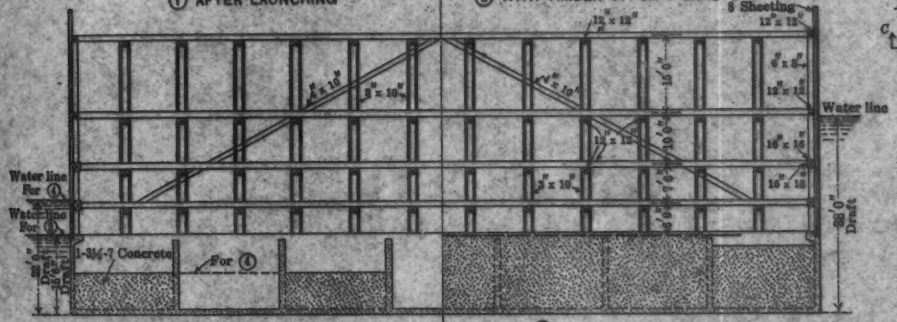
HARRIS ON
PEARL HARBOR DRY DOCK.

HALF SECTIONS OF REINFORCED CONCRETE BOX (CAISSON)



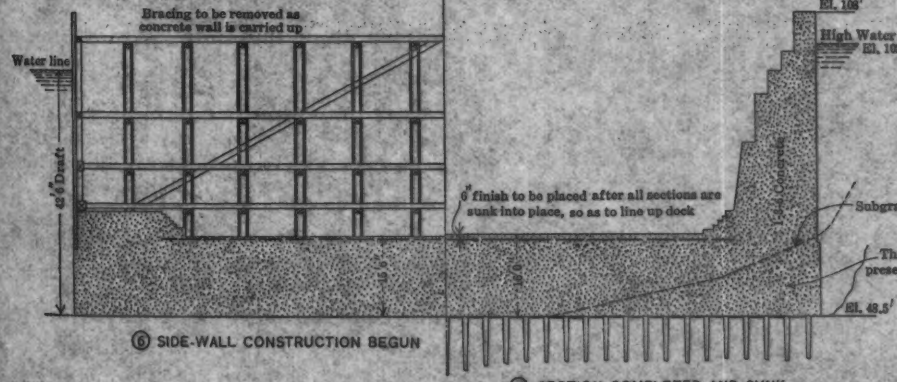
① AFTER LAUNCHING



② WITH TIMBER UPPER WORKS

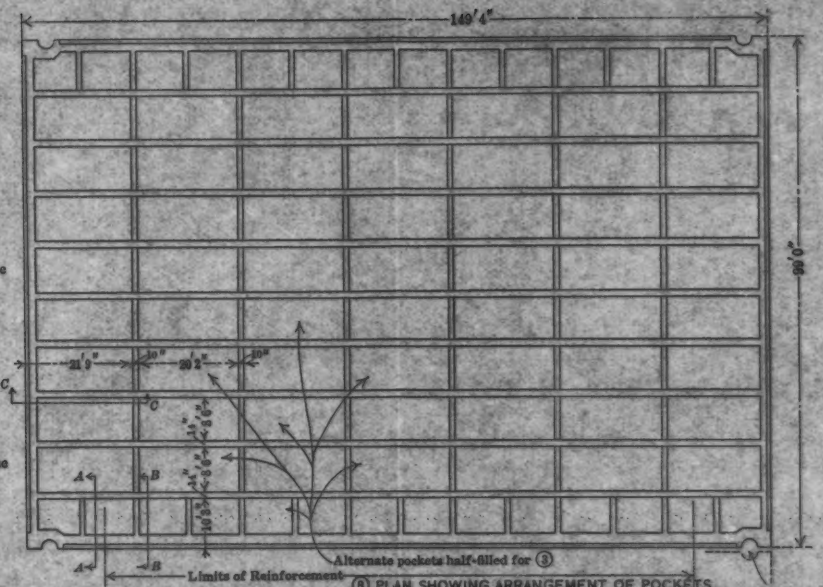
③ AND ④ POCKETS PARTLY FILLED

⑤ BOX COMPLETELY FILLED

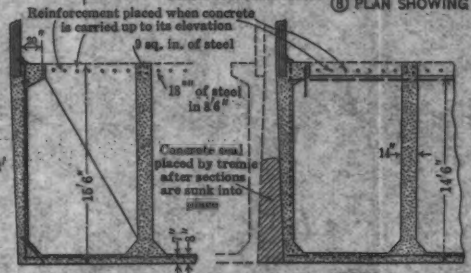


⑥ SIDE-WALL CONSTRUCTION BEGUN

⑦ SECTION COMPLETED AND SUNK ON PREPARED FOUNDATION

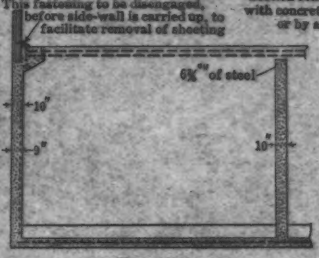


⑧ PLAN SHOWING ARRANGEMENT OF POCKETS



SECTION A-A

SECTION B-B



SECTION C-C

Section seal to be filled with concrete by tremie or by air lock

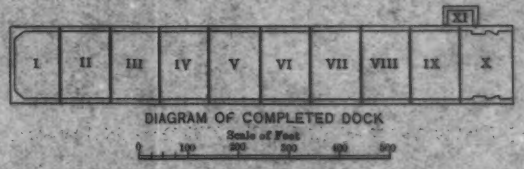


DIAGRAM OF COMPLETED DOCK

Scale of Feet 0 100 200 300 400 500

DETAILS OF SUCCESSIVE STEPS IN CONSTRUCTION OF PEARL HARBOR DRY DOCK

METHOD OF CONSTRUCTION PROPOSED BY F. R. HARRIS

and removing of the floating coffer-dam, and the successful lowering of these sections to place on a properly prepared foundation involve the greatest difficulty, and call for considerable ingenuity, especially in order to avoid great and severe stresses in landing a section and bringing it to an equal bearing over the entire foundation area. Even the greatest care and foresight may perhaps not forestall an accident, resulting in the loss of an entire section and the necessity for its subsequent removal. In his original recommendation, the writer suggested that the foundation be prepared by sawing off piles with an under-water saw arbor, and that each section be provided with a base of 2 or 3-in. soft pine planking, so that when settled to place on the piles, a more or less even bearing would be secured by local crushing. He understands that the plan now adopted contemplates cutting off the foundation piles about 1 ft. below sub-grade and covering the entire area with broken stone, to be leveled off under water. Mr. Harris.

LEONARD M. COX,* M. AM. SOC. C. E. (by letter).—This paper covers so thoroughly the history of the Pearl Harbor Dry Dock construction to date, as well as the difficulties encountered in connection therewith, that little remains to be said in the way of discussion, beyond congratulating the author on the novel design finally evolved for surmounting those difficulties, and on the very valuable contribution he has made to the literature of maritime engineering. Mr. Cox.

As two plans were mentioned in the paper as having been suggested by the writer, it may be of some slight interest to explain these plans, or schemes, for carrying out the work, in a little more detail, and to describe briefly the conditions which led to their submission. The first hardly deserves the title of "plan", as it never progressed beyond the sketch stage, and for the further reason that conditions at that time (June, 1911) were not such as to warrant serious consideration of a change either in design or in method of construction. It consisted in the building, on floating submersible platforms, of a number of reinforced concrete floating docks, of convenient length and of a width corresponding to that of the completed dry dock, the depth of pontoons and width of side towers to be such as would allow ample room for placing the lining, when in position, as a part of the completed structure. The floating docks were to be towed to their destined locations and sunk, by the admission of water, as adjoining sections of the dry dock. When landed on the previously prepared bottom, the interior chambers were to be filled with concrete, the joints between adjoining pontoons, or floating docks, were to be made with tremie-placed concrete, and the dry dock floor and side-walls completed to design lines after unwatering. It is unnecessary to state

* Brooklyn, N. Y.

Mr. Cox. that this plan would have involved great expense, a nice preparation of the bottom under difficulties, and the expenditure of much care and skill in the placing of the floating dock units.

In order to explain the second scheme, it is necessary to review briefly the conditions as known at that time. As stated by the author, the first attempt to unwater occurred about May 1st, 1911. This is fully described in the paper, and it will be recalled that the water level was reduced 16 ft. before evidence of bottom upheaval was observed, and, during attempts to unwater beyond this point, the bottom of the excavation was not ruptured. These circumstances led to the belief that if that particular section was typical of the entire area of

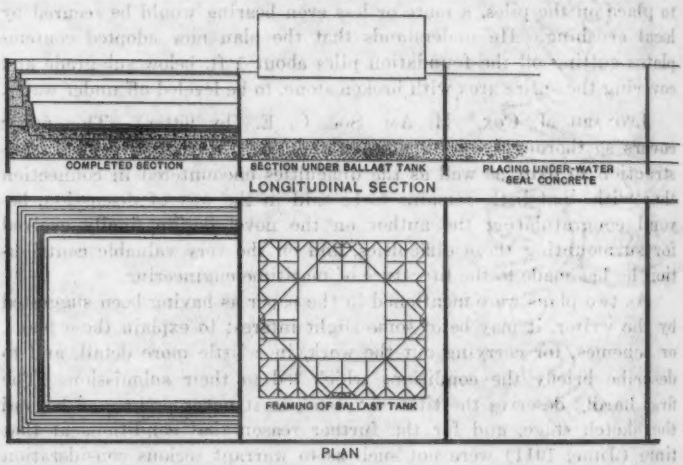


FIG. 22.

excavation floor, there existed below the dock prism a more or less impervious layer of such a thickness that its weight was equivalent to that of 16 ft. of water, a conclusion which was concurred in by the contractor's engineer.

As both the contractor's engineer and the Government engineer believed, from their experience on the work, that it was possible to drive piles 35 ft. into the underlying material, an agreement was reached by which additional compensation was provided for placing piles throughout the bottom, these piles to pierce the impervious layer on which the dock was to rest and pin into the material beneath. The notched heads of the piles were to project 4 ft. into the floor concrete, and the contractor was to be allowed to place not more than 8 ft. of sealing concrete by the under-water method. It was believed

that the anchorage value of the length of piling which would penetrate the material underlying the impervious layer, together with the weight of under-water-placed concrete and that of the impervious layer itself, would safely counterbalance the hydrostatic pressure which would exist when the excavation was unwatered. As a matter of fact, however, it was found to be impracticable, if not impossible, to drive piles of this length. Those driven in Section II, averaged a penetration of 21½ ft., with minimum penetrations as low as 7 ft., giving no anchorage value below the impervious layer. In this same section the concrete seal averaged 6½ ft. with a minimum of 3 ft. Of the entire section area, 55% contained piles which barely penetrated the assumed 19 ft. of impermeable material. Mr. Cox.

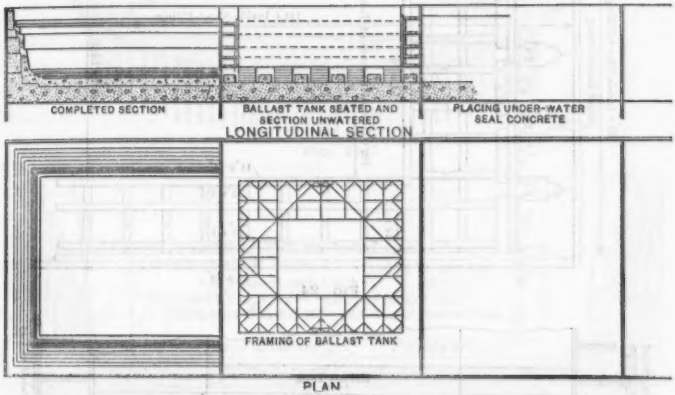


FIG. 23.

The writer was called in consultation in connection with the renewed studies and investigations following the second failure to dewater in February, 1913. Assuming that the failure was due to a lack of anchorage which prevented the unwatering of the partly completed sections necessary for the completion of the dock in the dry, and agreeing fully with the opinions of Mr. Noble and Mr. Harris in regard to the impracticability of using stone ballast, he devised a method of construction involving the use of water ballast. This method, he believed, and still believes, would have insured the completion of the dock by the original design, modified only by comparatively insignificant increases in the quantity of concrete involved and by the addition of certain reinforcing steel to compensate for the lost holding-down power due to short piles. He further believed that the adoption of such a plan would go far toward eliminating the risk of the original crib method, and would expedite completion.

Mr. Cox. In its details, the plan proposed no change in the contractor's bulk-headed sections, or in the preparation of his foundation by the driving of piles to the greatest penetrations possible to obtain. The heads of these piles, as called for in the original contract, were to be embedded

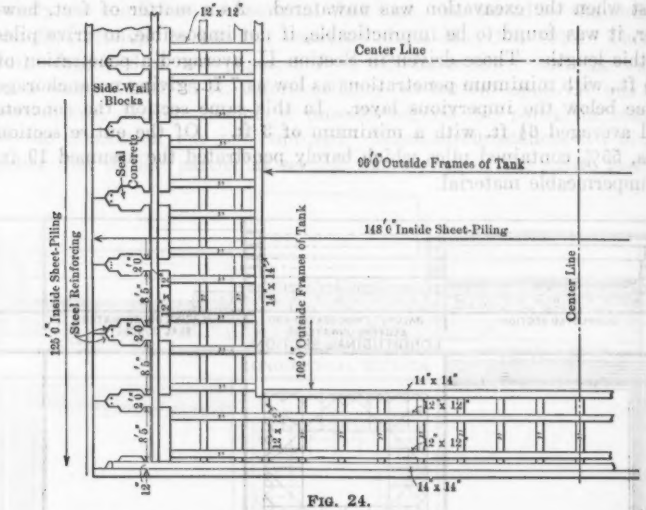


FIG. 24.

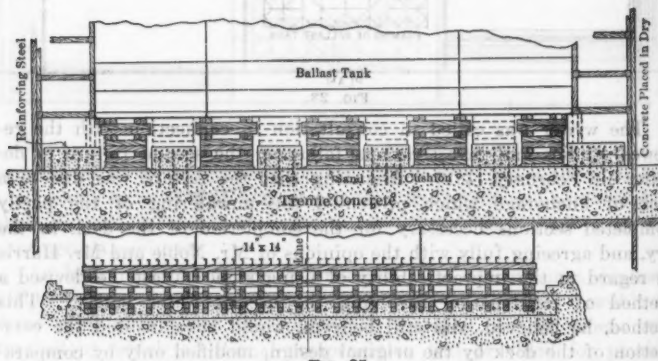


FIG. 25.

in from 11 to 8 ft. of tremie-placed concrete, as shown in Fig. 27. This under-water concrete was to be roughly surfaced at the sides for the reception of pre-moulded concrete blocks of large size. The blocks are shown in section in Fig. 27, and in plan in Fig. 24. They were to

be placed by the 150-ton floating crane already provided for the Pearl Harbor Station. After the piles, tremie concrete, and side-blocks were in place, a steel ballast tank was to be floated into position and sunk by the admission of water to a bearing on timber or metal keel-blocks arranged by divers across the dock section. Then, by pumping, the level of the water in the ballast tank was to be raised some 8 or 10 ft. above the level of the water in the excavation. With the tank resting on the under-water-placed floor, the next step in the operation

Mr. Cox.

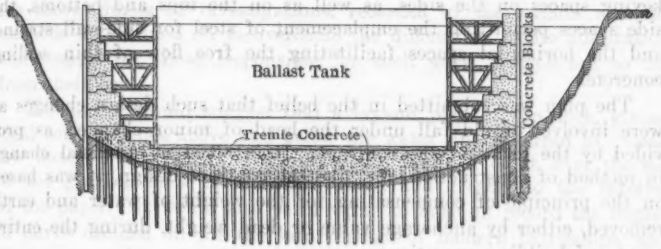


FIG. 26.

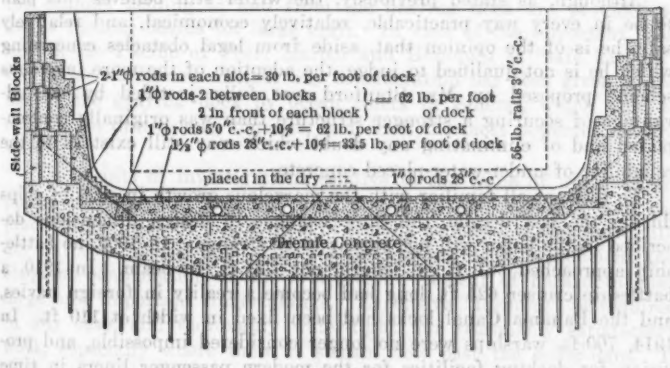


FIG. 27.

involved the unwatering of the space between the section bulkheads and the tank, shoring from the bulkheads or side-wall blocks to the tank sides as the pumping progressed. On the completion of pumping, the floor strips between the keel-block rows were to be completed in the dry, as shown in Fig. 25. The keel-blocks would then be shifted to these completed strips or ribs, and the space thus vacated completed in the same manner. The lower tier of shores between the ballast tank and the side-wall blocks was then to be removed, and the lining of the side-walls was to be placed, working from bottom to top. With

Mr. Cox. the entire section completed as described, water was to be admitted; then the ballast tank was to be pumped to light draft and floated over a shallow cut in the athwartship sheet-piling bulkhead to the next section, where the operations were to be repeated. The entire operation is illustrated in Figs. 22 to 27, from which it will be seen that ample provision was made for bonding the tremie concrete and that placed in the dry; also, that horizontal steel was introduced at the top of the floor-slab. The side-wall blocks were provided with female keying spaces on the sides, as well as on the tops and bottoms, the side spaces permitting the emplacement of steel for side-wall strains, and the horizontal spaces facilitating the free flow of thin sealing concrete.

The plan was submitted in the belief that such design changes as were involved would fall under the head of minor changes, as provided by the terms of the contract. It involved no essential change in method of construction, for, like the contract design, it was based on the principle of compensating for the weight of water and earth removed, either by anchorage value or dead weight, during the entire course of building operations.

Although, as stated previously, the writer still believes this plan to be in every way practicable, relatively economical, and relatively safe, he is of the opinion that, aside from legal obstacles concerning which he is not qualified to judge, the adoption of the more ingenious scheme proposed by Mr. Stanford was fully justified by the advantage of securing a stronger structure than was originally contemplated and of eliminating any doubt which may still exist as to the reliability of under-water-placed concrete.

To those at all familiar with the marvelous growth in sizes of ships during the last decade, the three changes in design dimensions described in the paper will not appear to be strange. In 1909, no battleship approached 581 ft. in length, nor 100 ft. in beam. In 1910, a battleship cruiser 625 ft. long had become a reality in foreign navies, and the Panama Canal locks had been fixed in width at 110 ft. In 1914, 700-ft. warships were no longer considered impossible, and provision for docking facilities for the modern passenger liners in time of war had become a real necessity. At the date of writing this discussion, battle cruisers 800 ft. long and of 100 ft. beam are seriously considered. The Department is to be congratulated on its broad policy of progress and on its determination to keep pace with the advances in every branch of science or art relating to its work.

Mr. Pretty. W. H. PRETTY,* ESQ. (by letter).—This paper, on perusal, conveys the impression of a vast piece of engineering research, from which the engineering world in general may read and learn. The plain

* Peterborough, Ont., Canada.

statement of difficulties met and to be overcome almost disarms criticism, and the determination to carry out the original scheme of building a graving dock in the face of such difficulties, was heroic. The writer's first impression, after reading the paper, was that the position is ideal for a floating dock in preference to a graving dock. The author, however, has already dealt definitely with this question, and no further reference is necessary. The work in hand is the building of a graving dock under extremely adverse conditions, and the thoroughness of investigations, recorded by the author, and the methods determined on to overcome the difficulties, excite admiration and respect for those responsible for such work, carried out so far from their base. Mr. Pretty.

The writer would suggest that a lay-out plan of the dock, giving local contour lines above and below the water line, referred to "mean low water" (to which it is presumed the depth of 35 ft. over the sill of the graving dock is referred) would add greatly to the interest of the paper. It would also be of value to have on record the degree of curve decided on for the safe passage of modern battleships, and the corresponding speed in knots; and, further, if any portions of the channel leading to and from the graving dock are provided to act as turning basins for large craft. In view of the important position in the Pacific of Pearl Harbor and Honolulu as "safe harbors", the writer would ask whether any consideration has been given to the use of the graving dock by ocean liners in general, which tend to outgrow in dimensions, tonnage, and draft the most powerful modern warships.

The results of investigations relative to the value of under-water-placed concrete by "tremies" are interesting and valuable, and the practical abolition of this method of placing concrete, except for auxiliary purposes, in the final plans for the graving dock, is sufficient and eloquent confirmation of the ever-present element of hazard in such large and important undertakings. Expensive "forming" is required in any case, and thoroughly water-tight, for reasonably satisfactory work, especially where there is stream flow past the coffer-dam. It is, however, a valuable aid for auxiliary purposes, and where the concrete thus placed is extraneous to the structure to be built, such as acting as a bottom seal for coffer-dams or to form a better bedding than the existing bottom provides. In the writer's opinion, a structural steel coffer-dam, with steel-plate shell, the whole of the lower part of which could be left permanently in the structure, would amply repay the first cost, in saving of labor, removing hazard, and in future security, and is an investment of the "safety first" order. The use of the "coffer-dam boat", described by the author, is unique, and suggests many possibilities for analogous use in building other structures and substructures, and the water tanks possess many advantages

Mr. Pretty.

for loading and unloading coffer-dams to meet the exigency of the moment.

The experiments on various concrete mixtures are full of interest, and seem to confirm the superiority of clean silicious sand in making "cement mortars". The writer's experience confirms this, and he is of the opinion that some form of surface chemical action takes place between the colloid silica in solution, in the newly-mixed, moist, cement mortar, and the clean quartz grains, probably in the nature of a process of segregation around these grains, which, when in close proximity, become cemented into a solid mass, improving with age; and when widely separated, or intermingled with other materials not amenable to this action, producing a weak concrete, deteriorating with age. We have instances of analogous segregation in the flints formed in chalk beds and calcareous sandstones; the kernels of the former are frequently pebbles, and occasionally have metallic nuclei of fossil origin. The writer has formed the impression from the latter portion of the paper that the use of silicious sand in making the concrete has been abandoned in the final plans for the dock, and that a form of so-called "puzzolan" mixture has been decided on. It would be interesting to know if the value of the latter for depositing in sea water is as great as generally reputed. Cements are often blamed when the mixing and "forming" are at fault, but there is no doubt, however, that each case must be dealt with on its own merits and from the standpoint of the nature of the water in which the concrete is to be deposited—be it salt, or brackish, or fresh water—and the paper certainly conveys the idea that this has been done in the work described.

It may not be without interest to mention here that the placing of under-water concrete has been reported to have been carried out with special dump scows, the concrete being placed in large, specially made, sail-cloth sacks, or bags, firmly sealed up before dumping on the site. The method has its possibilities, and doubtless its limits, when dealing with large undertakings.

The use of vertical tie-rods in the concrete near the outer faces of the walls is commendable. The writer does not recollect seeing any reference in the paper to the size of individual bars to be used for this vertical reinforcement, but may have overlooked this. Generally speaking, he is of the opinion that vertical ties near the faces of walls subject to hydraulic pressure should be individually of large sectional area with substantial anchorages, in preference to a large number of the usual type of small-section deformed bars, the latter being used merely as auxiliary reinforcement. The usual practice of artificially raising the elastic limit to an "excessive degree" by twisting and other deforming processes should be condemned.

Mr. Pretty.

The final plans, with all their completeness for a successful issue of the work, make one hesitate to make any comment save of appreciation for the thoroughness and ingenuity, combined with unique methods, which they show. The writer, however, would like to ask the author if the possibility of building the side-walls of the graving dock independently, received consideration, placing the dock floor later. This would seem to be, at first sight, a likely solution to the problem, the side-walls being built in alternate sections with coffer-dams, such as indicated by the writer in a previous paragraph, using an adaptation of the detachable coffer-dam boat of the author for the removable upper works, and leaving the lower part of the coffer-dams permanently buried in the concrete, the intermediate sections of the walls being built by the use of overlapping coffer-dams, checks being left in the base of the inner face of each side-wall into which the dock floor would lock. The dock floor would be laid in sections of convenient size, similarly, the ends of the coffer-dams in this case sealing with the inner faces of the side-walls of the dock. The tremie concrete in all cases acting as a seal only, would enable the actual true concrete of the structure to be placed in the dry, as indicated by Fig. 28.

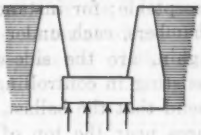


FIG. 28.

The adoption of main centrifugal pump units with vertical shafts, each direct-connected to a three-phase motor, forms an acceptable main pumping plant for a graving dock, where a shore or floating power station is available, and, as similar pumping installations have been already constructed elsewhere, it would be interesting to know if the 2 200-volt motors have proved all that can be desired under the trying conditions of pumping against varying heads, stopping and starting, to meet shoring requirements, etc., in the operation of a graving dock. At first sight, the writer would prefer 550-volt motors, reducing the size of the units, if necessary, and using more of them. Under the latter conditions, it should be possible, with three-phase motors, having suitable resistances in the rotor windings, connected up to the necessary controllers, etc., to have sufficient speed variation available to enable the motors to give out full brake horse power (approximately) throughout the whole period of pumping. The efficiency, from an electrical standpoint, might not be ideal, but it would have many practical advantages. It should not be forgotten, when considering the efficiency from a theoretical standpoint, that the pumps and motors may be idle for intervals extending into periods of weeks and months.

The reference to Artesian pressure and the tests carried out, make interesting reading, bringing to mind our school days and the teacher's

Mr. Pretty.

demonstration of the relative density of salt water and fresh water, in its application to engineering problems of to-day. Coming to the question of earth resistance vertically to loads placed on its surface, generally speaking, it may be taken that the average earth resistance over a large surface per unit area is greater than that obtained by tests made on relatively small isolated areas on the same site. In driving piles in "mud bottoms", it should not be forgotten that in such bottoms the hydrostatic uplift must be referred to the "lower ends of the piles themselves". A case came under the writer's observation some years ago where a large store shed (the floor of which was a concrete cap to a pile foundation in a mud bottom) rose 8 in. or more less than 2 years after the completion of the building. The site was on the marshes at the mouth of a river where the mud deposit was wet and of great depth. The rising was irregular, and the brick walls cracked in several places.

The idea of using a floating dock for the section bases speaks for itself, and deserves none the less credit on this account. A little more information about the timber floating dock to be used would be acceptable; for instance, are each of the five pontoons subdivided into chambers, each under independent control during docking operations? Again, are the side-walls also to act as independent pontoons for assisting in controlling docking operations? The writer would recommend that the ballast, which is to be placed on brackets on the inner faces near the top of the side-walls, be enclosed and under control, if such ballast is really necessary. There is nothing worse than moving ballast in the event of accidental heeling over from any cause whatever, whether the ballast is solid or fluid. To the writer it seems a pity that an appropriation was not secured for making this a small up-to-date floating dock of steel throughout, as it would prove a valuable auxiliary unit for use in Pearl Harbor and Honolulu after completing its work on the Pearl Harbor Graving Dock, for, in addition to its use as a floating dry dock for small craft, it would make a magnificent floating repair yard for ships in open water. It could also act as a floating power station and for salvage purposes, etc.

The use of structural-steel built-up frames, for embedding in the concrete of the bases, is a masterly solution of the problem of dealing with the dock floor, and forms a ready means of attaching stiffeners, etc., for the side-walls. The flat bottom adopted in the final plans for the graving dock is unquestionably the best for such a location and under such conditions as exist there. In the blasting operations carried out for the bottom bedding, is it not possible that the blasting charges may be too heavy? Smaller charges and more of them are better for foundations, as large charges disturb the bottom excavation very severely and very irregularly. The excess of weight of dock of

12 840 lb. per lin. ft. over hydrostatic pressure, referred to base of dock, does not appear to be very large, being apparently only about 2% excess per unit area of bottom of dock; this excess might be increased with advantage to stability. Mr. Pretty.

Concrete loaded with scrap structural-steel sections and punchings, etc., is occasionally useful, and particularly where a reduction in volume for a given weight is absolutely necessary, as occasionally happens in counterweights, etc. Blocks of such loaded concrete have been used by the writer, and, from actual measurements and weighing after manufacture and drying out, and where the concrete was apparently loaded to capacity with scrap metal as stated, the weight came out very approximately at 210 lb. per cu. ft. The use of such concrete in large quantities, however, would prove rather expensive.

Taking the condition of a heavier-than-water structure, it will be interesting to see the balance sheet of weights, and buoyancy, and the resultant "negative buoyancy" (resultant downward pressure) to the credit of the material of the dock, referred to the case where the dock is empty.

Considering, for the moment, the dock as a floating structure, with sealed ends, and "empty": let the volume of material of the dock $= V'$ and its average density $= \rho'$; the volume of the material of the dock below water line $= B$; and the volume of the material of the dock above water line $= C$.

$$\text{Then } V' = B + C \quad \ldots\ldots(1)$$

Let the volume of the empty space inside the dock "below" the water line equal A, and the total volume of fluid displaced equal V and its density equal ρ.

$$\text{Then } V = A + B \quad \ldots\ldots(2)$$

$$\text{and, } (A + B)\,\rho = (B + C)\,\rho' \quad \ldots\ldots(3)$$

$$\text{or, } A = V'\frac{\rho'}{\rho} - B = V'\left(\frac{\rho'}{\rho} - 1\right) + C \quad \ldots\ldots(4)$$

If we take $\Delta\rho'$ as the allowance for resultant downward pressure, then

$$A = V'\left(\frac{\rho' - \Delta\rho'}{\rho}\right) - B = \left(\frac{V'\rho'}{\rho} - B\right) - \frac{V'\Delta\rho'}{\rho} \quad \ldots\ldots(5)$$

$$\text{or, } A = V'\left(\frac{\rho' - \Delta\rho'}{\rho} - 1\right) + C$$

$$= \left\{V'\left(\frac{\rho'}{\rho} - 1\right) + C\right\} - \frac{V'\Delta\rho'}{\rho} \quad \ldots\ldots(6)$$

These equations are suggestive of useful approximations for mental checking as the work proceeds, both in design and construction.

Mr. Pretty. The writer takes this opportunity of thanking the author for bringing to his notice a paper of such unusual interest and wherein the difficulties met have been stated so frankly. It is, indeed, doubtful if any preliminary plans, without the experiences recorded, would have warranted such expensive methods of construction as those indicated in the final plans. To the professional onlooker, it represents a battle with natural forces in the cause of human progress, and one can only wish every success in the final results.

Mr. Baterden. J. R. Baterden,* Esq. (by letter).—This is one of the most interesting papers written on such a subject. In Great Britain we are apt to be afraid to put forth records of a work which has not been wholly successful, but, as the writer has pointed out more than once, the record of one failure teaches us more than many successes, as it enables us to provide against a similar failure in the future.

It is seldom that an engineer has to undertake the building of a dry dock in such a depth of water and in such bad ground as in this case, and the paper opens up so many questions and gives so many details that it is difficult to make useful remarks on it.

The writer has not much faith in concrete deposited under water as a successful method of forming the foundation of the floor of a dry dock, especially with such heavy hydrostatic pressure as in the present instance. A large proportion of the concrete deposited by the tremie method (as the writer would expect) was not water-tight, and some of it was "generally of poor quality". The author toward the end of his paper agrees as to the want of certainty of good water-tight concrete being deposited under water. One of the risks of depositing concrete by the tremie or similar methods is that the heavier particles of the mixture tend to settle in the bottom, and, unless divers are employed, there is no opportunity of having the mortar thoroughly mixed with the stone, and thus consolidated, as in ordinary open work. The consequence is that in many cases a heap of stones accumulates with little or no mortar surrounding them. Where, as in this case, it was found necessary to deposit concrete in this manner, a richer mortar and of larger volume than is customary in ordinary work should be used.

The writer's experience in the use of "plums" in concrete for dry dock work has not been satisfactory. Theoretically, they are all right, but his difficulty has always been to ensure that the "plums" are satisfactorily placed. As long as they are completely surrounded by a sufficient thickness of water-tight mortar, the work is quite satisfactory, but the difficulty is to get this regulation adhered to. Even with the strictest supervision there is a risk of a heap of stones being deposited—for, of course, it is to the contractor's advantage to get

* Newcastle-on-Tyne, England.

Mr. Baterden.

as many "plums" in the work as possible—or of their settling close together and not having sufficient mortar around them, thus forming a sieve for water to penetrate. The extra weight given to the concrete by their use is very little; the saving of cost is certainly appreciable. Where "plums" are used, the stone ingredients of the mixture should not be more than would go through a sieve of, say, from 1 to 1½ in. square.

The writer quite agrees with the author's statement that mortar mixtures for water-tight concrete should be not more than 1 part of cement to 2 parts of sand. This produces a volume of about 2.1, so that the volume of ingredients for the concrete, allowing 45% for voids, should not be more than 3½, which will allow a safe margin of mortar to fill the interstices and surround the stones; if he had a Government behind him, the writer would feel inclined to make the proportion of broken stone only 3, thus ensuring a harder and more water-tight concrete.

It would appear, from the revised plans, that the idea of an intermediate caisson has been abandoned; this arrangement, in the writer's experience, has not been worth the extra cost.

It is a question whether a dry dock, 1 000 ft. long, is required in such a location. It is very unlikely that such a length will ever be required for Admiralty purposes; and, although it is true that the Panama locks are more than 1 000 ft. long, their case is different, as they provide for allowing two or possibly four of the present-day large vessels to pass through at the same time, as well as for the extreme length and beam of ships of the future. The 1 000-ft. ships of the future will be comparatively few, and every 50 ft. of shortening of dock, in ground such as that at Pearl Harbor, means a considerable saving.

The particulars given of bearing power and adhesion of piles in the ground are very interesting. As an instance of the weights which piles will carry, even in very soft ground, the writer, many years ago, supervised the test on four piles, 3 ft. from center to center, driven 25 ft. into the ground, and having on top a platform 6 ft. square to carry the load. The foundation at this place consisted of soft mud and muddy sand for a depth of about 180 ft. A load of 72 tons was placed on the platform, 18 tons per pile, which was much more than the piles in the adjoining ground were to carry. The following was the total settlement of each of the four piles, 6 days after the load had been placed on the platform: 1.08 in., 1.32 in., 3.96 in., and 3.12 in.

The greatest loading test on a single pile, of which the writer is aware, was one supervised by himself more than 20 years ago to test the ground for a battleship berth. The pile was of sawn pitch pine, 42 ft. long, about 12 in. square, and driven 37 ft. into the ground, which consisted of clays of varying degrees of stiffness, finishing in

Mr. Baterden.

a soft leafy clay. The weight of the hammer used was 5 600 lb., its fall was 5 ft., and the finishing sets were ¾ in. This pile actually carried 97 tons before commencing to sink, and then sank quickly and steadily.

It is a moot point whether piling is necessary as a foundation for a concrete dry dock, even in soft ground. The writer has had experience in dry dock construction in the worst of ground, where no piles were used, and the result was quite satisfactory. His experience with the jet process of pile sinking in mixed ground has been much the same as that of the author, and he believes it is only satisfactory where the subsoil is wholly in sand or fine material of a uniform character.

The new designs and method of carrying out the work might be called heroic, and appear to have been carefully thought out. The straight girder, as decided on, is certainly the best. Whatever slight advantages might be gained in strength by the arched form, and the lesser quantity of concrete required, would be more than counterbalanced by the almost impossibility of getting the required curved profile dredged accurately in such a depth of water. The writer hopes, and quite believes, that the work will be brought to a satisfactory conclusion.

Mr. Bellamy.

HERBERT E. BELLAMY,* ASSOC. M. AM. SOC. C. E. (by letter).—This admirable paper is most interesting, and is certain to hold the attention of a very large number of the members of the Society, as all are hoping for the successful solution of the manifold difficulties and problems which the construction of the Pearl Harbor Dry Dock has involved.

To the mind of the colonial engineer, the question of a dry dock of the floating type for economy naturally presents itself; but it would be useless to consider this point further, as the author specifically states:

"All questions as to the type of dock to be provided are in this case beyond discussion, inasmuch as the Congressional authorization for the work distinctly directs the construction of 'one graving dry dock'."

It is hoped, however, that the author will supply additional information on the subject that he has submitted for discussion, namely, the dredging operations, completed in 1912, connecting the harbor with the sea, as described under the heading "Location". It is stated that these operations required the removal of 4 645 000 cu. yd. of material, throughout a channel 5 miles long, at a cost of $3 296 000. It would be of very great interest to know the type, size, and number of dredges used, the method of working, delays, etc., and full particulars while the dredges were engaged over the entrance bar where there was a heavy sea swell at all times.

* Melbourne, Victoria, Australia.

The progress of the works under discussion will be closely followed, especially by engineers resident in Australia, in which country extensive naval works are in contemplation. Mr. Bellamy.

B. C. LAWS,* ESQ. (by letter).—The writer regrets being unable to enter into a technical analysis of this interesting paper, although he has been able to peruse it sufficiently to realize the great instructive value it must have for all engineers concerned with work of this type, who may have the opportunity to read it. Mr. Laws.

The temporary setback, due to the collapse of the coffer-dam on February 17th, cannot but have had a depressing effect on those engaged in the design and construction of the dock, but such an interruption to work, unpleasant as it undoubtedly is at the time, is often a blessing in disguise; it sets us thinking, and frequently new problems or new phases of the old problem, which otherwise would have remained latent, are evolved, the solution of which has a usefulness and application which reaches beyond the work in hand.

Congratulations should be extended to the author and those associated with him, who, after careful and exhaustive analysis of all the factors in the case, have arrived at a course of procedure which bids fair to terminate with great success.

EDWARD BOX,† ESQ. (by letter).—The writer has been much interested in this very lucid description of the attempted construction of a graving dock at Pearl Harbor. Mr. Box.

Whether one regards the undertaking as a reasonable attempt to "direct the great sources of power in Nature for the use and convenience of man", or as a badly conceived scheme which disregarded the laws of Nature, one cannot but admire the very candid way in which the subject has been presented in this paper which should be of estimable value to the student of engineering, and a contribution of more than ordinary value to the world's records of engineering works.

After reading the carefully prepared description of the locality in general, coupled with the particular nature of the site, one cannot help but ask, how was the decision arrived at, which caused Congress, in May, 1908, to specify particularly a graving dock?

As a representative of Great Britain, the writer had the honor of reporting to the International Congress of Navigation, held at Philadelphia in 1912. The subject assigned to him was "Means for Docking and Repairing Vessels". As he then discussed the relative merits of the two systems of dry docks in vogue, *viz.*, graving docks and floating docks, it is not necessary at this time to repeat the views then expressed. Those who are sufficiently interested in this subject will be familiar no doubt with the papers read before that Congress. The

* Hull, England.

† Newcastle-on-Tyne, England.

Mr. Box. writer, however, will only repeat here that it is generally recognized that there are two equally efficient methods of laying dry, for painting and repairing purposes, the larger types of ships, whether they are ships of war or of commerce, *viz.*, the graving dock and the floating dock.

The site, as described in the paper, is one which seems particularly to favor the adoption of the floating dock. The author discusses the question as to choice of type of dock by reference to the Congressional authorization for a graving dock, but Congress went further than that in its instructions, by limiting the cost to $2 000 000, and one becomes interested to know how that amount was arrived at, and to ask whether a graving dock would have been insisted on, had it been known that there was likely to have been so much uncertainty in the cost of its construction.

If, however, we regard the undertaking purely from the point of view of an engineering problem to be achieved at all costs, as we are asked to, the relative merits of the two kinds of docks becomes a separate question, although none the less an important one, and the only feature that remains for discussion, is how to obtain the end in view. The past history has been a chapter of failure for the reason that the difficulties of the undertaking—the writer thinks there can be no doubt about it—were not properly realized when the method of construction was decided on.

In a case, some years ago, where a project was under consideration for a Government graving dock on the Atlantic Coast, under conditions somewhat similar to those at Pearl Harbor, the writer was interested in the preparation of the estimates. The estimated cost was more than $5 000 000. The figures were based on harbor work prices, and, therefore, were probably fairly reliable. This estimate was for a dock considerably less in dimensions than that being built at Pearl Harbor. The project was abandoned, and a floating dock was constructed at much less cost.

As an engineer, the writer has long cherished the desire to build a dock on a site presenting difficulties such that the usual methods of construction would no longer suffice. He has never known a case, however—although there may have been one—such as Pearl Harbor presents, where a graving dock has been attempted. What he has had in mind is a modified design for a floating dock to be sunk on a dredged site and then converted into a concrete graving dock by sectional treatment.

To his mind, a structure complete within itself, although built first in sections, sunk into place, and then treated sectionally, is the preferable method, where practicable, of dealing with a problem such as Pearl Harbor would appear to present. In this way the question of compressibility is practically eliminated, and equal loading throughout is obtained.

Mr. Box.

With regard to depositing concrete, the best concrete obtainable is that which is deposited in the dry and flooded on initial set. In graving dock work, the writer never uses concrete of less strength than 5½ to 1 of cement, which would be considered a weak mixture in the present case. The proportion of sand to ballast, of course, depends on the grading of the particular materials used.

The site for the Pearl Harbor Dock has been badly tampered with and, for this reason alone, whatever method is adopted, further difficulties must of course be expected, the worst feature of which appears to be the piling of the ground. With the necessary funds, tenacity of purpose, and under the guidance of practical engineers of matured experience, it is possible that some day the Pearl Harbor Dry Dock will be an accomplished fact.

The writer cannot close his remarks without expressing sympathy—a sentiment almost unknown with reference to engineering work—with those who have been laboring so hopelessly for years on what must have been regarded by many as an impossible task.

Mr. Stanford.

H. R. STANFORD,* M. AM. SOC. C. E. (by letter).—The writer is extremely grateful to those who have been interested enough to discuss his paper, and extends to them his thanks and appreciation.

No attempt will be made in this closing discussion to consider any remarks or opinions, expressed in preceding discussions, which are controversial in character, inasmuch as the paper only purports to be a record of facts and conditions. Careful effort will be made, however, to answer all questions which have been raised, and to supply the additional data or information requested.

In reply to questions raised by Mr. Taft, it may be stated that dilution with fresh water of the sea water contained within the cofferdams, in order to reduce the saline content and to avoid the deleterious effects of salt water on freshly placed concrete, was considered, but the idea was necessarily abandoned because of the limited supply of fresh water.

Experimental cubes of tremie-placed concrete, measuring 6 ft. on the side, were made with one brand of German cement and two brands of American cement, in order to determine, if possible, whether the use of a special brand of cement for tremie concrete would be warranted. These experiments indicated that "Santa Cruz" cement, which had been approved for the work, gave results as good as those obtained with the other test cements, and that the kind of cement which had been used was not chargeable with the trouble experienced with the tremie concrete.

The cement approved for use in the dock is the "Santa Cruz" brand, made at Davenport, Cal., subject to the standard Navy Depart-

* Philadelphia, Pa.

Mr. Stanford.

ment specifications, from which the following, relative to chemical requirements, is quoted:

"The cement shall be the product obtained by finely pulverizing clinker produced by calcining to incipient fusion an intimate mixture of properly proportioned argillaceous and calcareous substances, with only such additions subsequent to calcination as may be necessary to control certain properties. Such additions shall not exceed 3% by weight of the calcined product.

"In the finished cement, the following limits shall not be exceeded:

"Loss on ignition for 15 min.....4.00%
Insoluble residue.....1.00%
Sulphuric anhydride (SO_3).....1.75%
Magnesia (MgO).....4.00%"

The importance of the proper mechanical mixing of the concrete in a batch is fully recognized, and steps have been taken to insure that there shall be no failure in this respect due to mixing for too brief a period or at too rapid a rate.

The experimental investigations in the use of tremies, which were conducted in connection with this work, did not give much consideration to methods of freeing the tremie from water when it was being charged at first. This apparent neglect was due to the fact that the straw pillow or plunger inserted in the tremie when the process was started practically prevented the salt water from entering all but the very first portion of the concrete charge and this part of the concrete, under any circumstances, would necessarily become impregnated with salt water when issuing from the tremie to begin the filling of the form. The straw pillow remained in the bottom of the form and was so compressed as to be unobjectionable under these conditions; an elaborate or expensive plunger was not considered justified.

In reply to a question by Mr. Pretty: a contour survey of the dock site, made before construction work was started, showed that the surface of the ground sloped quite uniformly from an elevation of approximately 14 ft. above mean low water at the head of the dock to an elevation approximately 10 ft. below mean low water at the entrance.

For a distance of approximately one mile from the open sea, the entrance channel to the Naval Station is straight, and has been dredged to provide a least width of 600 ft. The remainder of the channel has a least width of 500 ft. in the straight portions, and somewhat greater widths in curves. The minimum radius of the dredged channel is 1 600 ft. The safe speed for deep-draft vessels through this channel is from 4 to 6 knots per hour. There are large areas within the harbor, in the vicinity of the Naval Station, over which the natural depth of water is much in excess of that required by naval vessels, and this rendered the dredging of turning basins unnecessary.

Mr. Stanford.

The history of dry-dock construction in America is apparently much the same as that in other countries. With the authorization of each new large dock, effort was made to anticipate the growth of ships and to make the dock large enough to admit the largest vessel which would probably be built; due to the very rapid increase in the dimensions of ships, however, it has frequently been the case that when the docks were completed, usually after protracted delays from one cause or another, they had actually been outgrown during the period of construction. When dimensions were fixed for the locks of the Panama Canal it was expected that those dimensions would establish limits in the design of new ships for both the naval and merchant services, and, when the dimensions of the Pearl Harbor dock were made to accord with the Panama locks, the designing engineers had satisfaction in the thought that this would be one dock at least which would not be outgrown.

The revised plans do not provide for the use of any imported silicious sand in making concrete for the body of the dock, except for the small quantity of tremie-placed concrete required for making the seals between adjacent sections. It is expected that the rich mixtures specified will result in making a practically impervious concrete which will insure the mass against depreciation from chemical changes and disintegration and will efficiently protect the embedded steel.

The vertical reinforcement rods near the outer vertical faces of the side-walls of the dock are $1\frac{5}{16}$ in. in diameter and 17 ft. long, and are 18 in. apart. The specified range of ultimate tensile strength of these rods is from 55 000 to 70 000 lb. per sq. in., which is a safe limit, without making any allowance for increase resulting from the mechanical process of twisting or deforming. The sectional area of all embedded steel is made heavy as a protection against corrosion losses.

The method of building the side-walls first and then constructing the bottom between the walls was given very careful consideration, but the cost and difficulty in arranging details of construction, which would avoid the necessity for any considerable quantity of under-water placed concrete in building the central portion of the dock, led to the abandonment of the idea.

There are three large naval docks, which have been in operation for some time, each of which is unwatered by four vertical-shaft pumps operated by 2 200-volt motors. No trouble whatsoever has been experienced with these motors, either as a result of the varying head on the pumps, or in starting or stopping the motors. The suggestion that the motors might advantageously be equipped with slip-rings and controllers, in order to obtain a certain degree of speed regulation, is a point well taken; with a 10% control, it is practicable to regulate the power input into the motor so that it will remain almost constant

Mr. Stanford.

during the entire range of pumping, and this is a very material item, considering that the dry-dock load imposes the peak-load condition on the power plant, and that maximum motor efficiency is of little consequence, because of the brief and infrequent intervals of pumping. It has been found in service that four pumps for one dock is a very desirable number, to provide for the regulation of unwatering in docking a ship, and also for break-down service; to increase the number with corresponding reduction in unit capacity would tend to add to the cost of the installation, on account of the expense of their valves and connections.

Each of the five pontoons of the floating dock is divided by a water-tight center bulkhead into two water-tight compartments; each of these compartments is further subdivided by swash bulkheads into four sub-compartments. All pumping is performed from the side-wall compartments, and each pontoon compartment can be pumped from either side-wall.

The small floating dock is being provided by the contractor as an item of plant necessary for the construction of the graving dock. The Navy Department fully recognized the great value which this floating dock would have as a permanent feature of the station's equipment on the completion of the graving dock, but, in view of the difficult and more or less hazardous character of the service which will be performed by the dock for the contractor, it was thought best to make the floating dock specifically an item of plant, in order to avoid the possibility of question as to responsibility in case of trouble.

Great care is being exercised in conducting the dredging operations, and in cutting off the tops of piles to avoid injury to the remaining piling which will form part of the prepared foundation for the dock. If any injury is suffered by this piling, it will be necessary to replace the injured piles with good ones.

The following are the estimated values, per linear foot of dock, of various elements in the completed structure, which will act in the nature of weights or ballast, in addition to the estimated force of 12 840 lb. due to the weight of concrete, to insure a positive downward resistance to flotation or uplift:

(*a*)	Due to the weight and structural moment of the reinforced concrete ties in the surrounding crane track..	1 360 lb.
(*b*)	Weight of soil on the narrow outstanding shelf in the outer face of the side-walls.....	11 160 "
(*c*)	Probable average weight of concrete at 154 lb. per cu. ft., instead of 150 lb.....	14 080 "
(*d*)	Weight of blocking on floor of dock.....	1 800 "
(*e*)	Frictional resistance of earth against side-walls.....	36 000 "
		64 400 lb.

Mr. Stanford.

Referring to the statement of Mr. Baterden that "it is a question whether a dry dock, 1 000 ft. long, is required in such a location", it should be noted that a dock having a lesser length would not comply with an implied requirement of the Act of Congress under which the dock is authorized. This reason is mandatory, aside from any consideration of military need.

The writer is in agreement with Mr. Baterden as regards the necessity, in general, for foundation piling for a concrete dock, even when constructed in soft ground. An essential reason for the great thickness of concrete in the floor of a graving dock, built in a reasonably favorable soil, is that it gives the dock sufficient weight to resist flotation; in other words, the dock weighs little more than the water which it displaces, and therefore imposes on the underlying soil a weight which is only a little more than that of the water which would naturally flood the completed excavation for the dock, and probably much less than that of the soil which was removed in making the excavation, and which in original formation had been safely supported for ages. Under these conditions, and in ordinary cases, there can be little need for piling if the dry dock structure is sufficiently strong to distribute its weight over relatively small soft spots in the bottom of the completed excavation, and the use of piling under such circumstances is in the nature of a precautionary measure or factor of safety.

Dredging for the channel was begun at the outer entrance, in water from 15 to 25 ft. deep, by using a steam lumber schooner fitted with hoppers and equipped with a 14-in. centrifugal pump, connected to a 20-in. suction pipe, and driven by a 275-h.p. steam engine. The suction pipe was rigged from an A-frame forward. This machine handled sand, loose coral, and mud very effectively in shallow water. As the depth became greater and the softer material was removed, a clam-shell dredge, equipped with a 4½-yd., 13-ton bucket, was put into use. This machine handled mixed materials as encountered, but did not remove successfully coral in ledge formation nor partly decomposed lava rock. For excavating these hardest materials a drag-scraper dredge, equipped with a 3-yd. bucket, was used; the bucket was sunk at the greatest possible distance from the dredge, which was moored with anchors, the water being too rough to permit the use of spuds, and was drawn in by a cable rove through a sheave at the stern of the dredge; the bucket was hoisted when a few feet forward of the hull. This dredge operated successfully on the hardest materials, and at the greatest depths, and was especially successful on the clean-up in cuts from 37 to 38 ft. deep, with a minimum of 35 ft.

A 16-in. suction-dredge, without cutter head, and operating from moorings, was used at the inner sections of the work. This dredge did not operate successfully in a seaway, and was used more extensively

Mr. Stanford.

on the less exposed parts. Attempts were made to operate both suction-dredges and a dipper-dredge from spuds in the entrance fairway, but there was always enough swell, even in the stillest weather, to render the use of spuds impracticable.

The dredging of the channel was begun on August 23rd, 1909, and completed on January 30th, 1912. During this period the weather conditions were generally favorable, with no heavy tropical storms, although the dredges were frequently withdrawn to shelter on account of threatening weather. For months at a time the weather was fair, with trade winds springing up at about 10 in the morning and dying away between 4 and 6 in the afternoon. The evenings, nights, and early mornings were usually still. At the outer entrance, due to the trade winds, a fairly heavy swell was running at all times, and, as the deepening progressed, this extended farther and farther into the channel. The hard material in the entrance was broken up by drilling and blasting before cleaning up with the drag-line dredge. After the finished depth was nearly reached, considerable blasting was done without drilling, charges of explosives being placed on submerged boulders or ledges and fired by electricity. These blasts were quite effective, but only over a limited extent, and to a very limited depth.

The questions raised by Mr. Box are very interesting. In stipulating that the dock should be of the graving type, Congress was undoubtedly influenced by the fact that a dock of that type, after being completed, involved practically no expense for repairs and upkeep, whereas those costs for a dock of the floating type, with its mooring and approaches, are continuous and large. Although Congress is apparently willing to appropriate liberally to provide for the first cost of public works improvements, there is great difficulty in obtaining annual appropriations for maintenance and repair. Estimates for new works, therefore, are very apt to be larger than would otherwise be the case, to provide for very permanent and durable construction. As a business proposition, it would frequently be economical, in the long run, to spend less to begin with and a little each year for maintenance, so planning that the interest on the saving in first cost would more than equal the maintenance expense; but, knowing that there will be difficulty in obtaining the maintenance fund, there is a tendency to adopt the more extravagant method.

Floating docks have been found very efficient and satisfactory in the docking of our naval vessels, and there can be no question as to the practicability, value, and usefulness of docks of that type. The writer will frankly concur in the opinions expressed by Mr. Pretty, Mr. Bellamy, and Mr. Box, that the natural depth of water in Pearl Harbor, combined with the unfavorable soil conditions found at the

Naval Station, are strong arguments favoring the selection of the floating rather than the graving dock for that place. Mr. Stanford.

More thorough borings and examination of the ground conditions at the dock site before beginning operations would probably have obviated at least a part of the difficulty and expense which have been incident to the work. Such examinations, together with careful preliminary investigations of the material and labor resources in the vicinity, would also have shown that the first estimate of cost of the dock was quite inadequate. The explanation of the failure to obtain these data lies in the fact that there was a sudden realization of the importance of quickly developing this base, coupled with the altogether too frequent error of laymen in failing to recognize properly the importance and need of the engineer for such service.

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Paper No. 1355

THE HYDRAULIC JUMP, IN OPEN-CHANNEL FLOW AT HIGH VELOCITY*

By Karl R. Kennison, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. B. F. Groat, H. B. Muckleston, Frederic P. Stearns, Carl B. Andrews, R. D. Johnson, Mansfield Merriman, E. G. Walker, Edward W. Bush, H. F. Dunham, Robert F. Ewald, L. D. Cornish, Caleb Mills Saville, and Karl R. Kennison.

Synopsis.

This paper presents an analysis of the hydraulics of the turbulent discharge below a spillway dam, where the so-called "jump" frequently occurs. A knowledge of the hydraulic principles involved should enable destructive high velocities and turbulence to be avoided, or intelligently provided for, in the design of flumes, dam foundations, etc.

The arrangement of the paper is as follows:

1.—Introductory.

2.—Conclusions.

The interesting feature is that in every open channel, except at controlling sections where the discharge is a maximum, there is, in addition to the existing water level, another level at which the same quantity of water might be flowing, under the same head. These two "alternative stages" should be recognized in the design of all structures for controlling the flow of water. The hydraulic jump is merely the turbulent passing between these two stages.

* Presented at the meeting of November 3d, 1915.

3.—Mathematical Basis for Conclusions.
(*a*) Relation between Depth, Head, and Discharge.
(*b*) Maximum Discharge at Controlling Section.
4.—Examples of the two Alternative Stages.
5.—Examples of the Hydraulic Jump.
6.—Destructive High Velocity below Spillway Dams.

1.—Introductory.

When water is discharged into a flume through a contracted gateway and under a considerable head, it sometimes continues to move in a thin sheet at a high velocity along the bottom of the flume for several hundred feet. Then it suddenly becomes turbulent and forms what is called a "hydraulic jump", the surface level down stream from this point being much higher than that of the approaching high-velocity discharge; or, when water flows over an ogee dam and out on a smooth apron it sometimes continues in a thin sheet, having a surface level far below the normal level of the river a little farther down stream, until it suddenly changes into a tumbling mass, rising to the normal river level by this "back roll" or "hydraulic jump". An excellent illustration of this is seen in Fig. 1, taken from a photograph, kindly furnished by the United States Reclamation Service, of certain flood conditions over the Granite Reef Dam.

This phenomenon sometimes becomes of great practical importance, and has been investigated mathematically by the writer at the request of John R. Freeman, M. Am. Soc. C. E., because of its interest in connection with the design of two important dams under widely different conditions in different parts of the country. In one of these cases it was desirable that the back roll or hydraulic jump should not be pushed far down stream from the foot of the ogee, off from a concrete apron which protected a clay river bed from scour; in the other case it was desirable that the back roll or hydraulic jump, with its violent surges, should not be pushed down stream so far as to interfere with the draft-tube exits of the power-house.

The problem does not appear to have received proper attention in the textbooks. No exhaustive mathematical analysis is attempted here, but merely an explanation of the peculiar conditions of flow which make the jump possible. The conclusions are as follows:

2.—Conclusions.

In the case of water flowing in an open channel on a steep gradient there are certain controlling sections which throttle the flow and determine the quantity of the discharge, that is, certain points where, for the given head and channel depth, the discharge is a maximum. If the contraction which causes this throttling of the flow is sufficiently gradual—for example, a submerged dam with smooth gradual approach and get-away—it can be shown that the depth of water at this point is theoretically two-thirds of the total head measured from the channel bottom or dam crest up to the hydraulic gradient, and the discharge per foot of length, therefore, should be $3.09\ H^{\frac{3}{2}}$.

At other points than at the controlling sections, however, the depth of water is not necessarily determined by the quantity discharged and the available head, but also by the channel conditions; because, for a given quantity of water flowing and a given head or elevation of hydraulic gradient, there are two possible surface-water levels which we will call alternative stages. The upper stage is the normal level in an ordinary stream, and for very low velocities is practically coincident with the hydraulic gradient. The lower stage is that ordinarily taken by water discharged at high velocity from an orifice or below a spillway dam. This is the more unstable of the two levels, due to the friction of high velocity on the channel bed. In other words, it can be shown that in any open channel, except at controlling sections such as just referred to, there is, in addition to the existing water level, another level at which the same quantity of water might be flowing with equal steadiness and under the same head or elevation of hydraulic gradient.

As these two definite alternative stages are the only possible ones under the existing head for smooth undisturbed flow, the stream must stand at one of these two levels, that is, water flowing in a smooth channel of uniform section must continue to flow at its existing stage, whichever one that happens to be, until, either due to a change in the channel bed or after sufficient loss of head in friction, it encounters a controlling section where the two alternative stages merge into one, and the depth is two-thirds of the total head. Below this controlling section the two possible stages again separate. At such a point as this the water level may change, without disturbance or interruption of the steady flow, from upper to lower stage, or from lower to upper

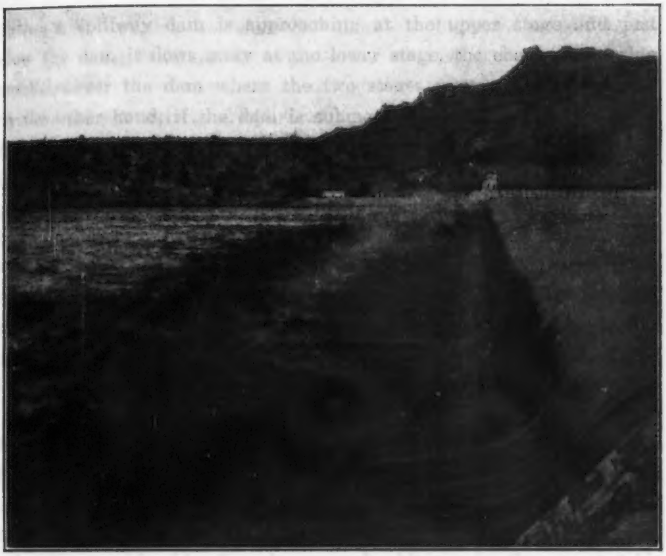


FIG. 1.—THE GRANITE REEF DAM IN ACTION.

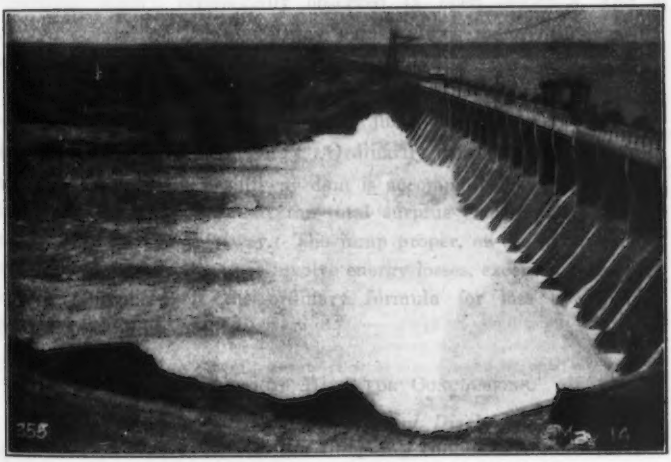
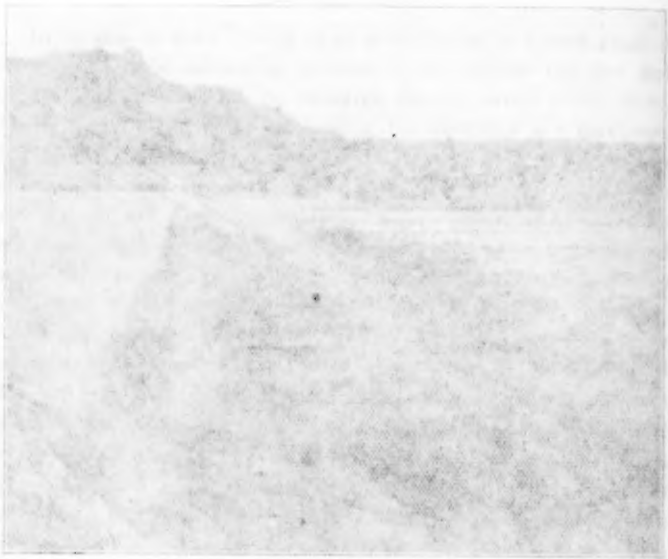
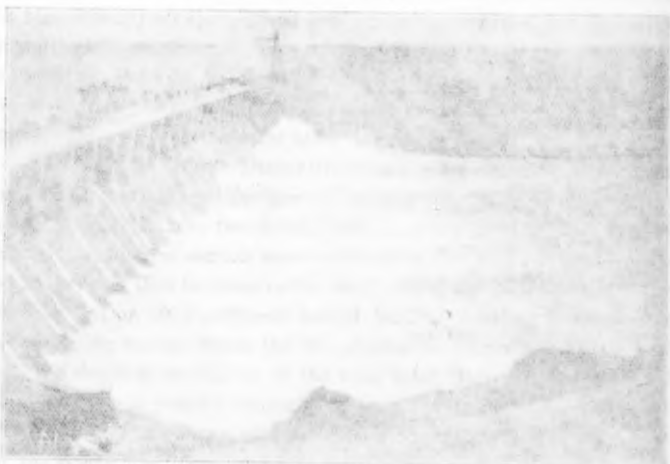


FIG. 2.—THE BASSANO DAM IN ACTION.





stage, or may continue at the same stage. For example, the water behind a spillway dam is approaching at the upper stage and just below the dam it flows away at the lower stage, the change occurring smoothly over the dam where the two stages were merged into one. On the other hand, if the dam is submerged by back-water almost as high as the up-stream pool, the surface may simply dip down locally at the dam where the depth is two-thirds of the head. In this case the upper alternative stage is maintained throughout, below as well as above the dam. It can also be shown that under certain circumstances the flow over a dam may theoretically be reversed, the dam facing down stream, and the water apparently running up hill—simply a case of passing from the lower to the upper stage smoothly, over a gradual controlling section where the depth is equal to two-thirds of the head.

In such phenomena the presence of the controlling section tends to eliminate any disturbance in passing from one level to the other, so that the existence of the two alternative stages is not noticed; but water flowing at the lower high-velocity stage and suddenly encountering obstructions which tend to destroy its velocity may rise suddenly, and with considerable disturbance and eddying, to the more stable upper low-velocity stage, and this phenomenon is the so-called "hydraulic jump", occasionally observed in open channels, and a common occurrence below spillway dams. It is merely the passing between the two alternative stages.

In this phenomenon the only energy loss is that due to the accompanying disturbance and eddying, the jump proper merely transferring kinetic into potential energy. Ordinarily, however, this hydraulic jump occurring below a spillway dam is accompanied by such violent disturbance and eddying that the total surplus energy in the water may be destroyed in this way. The jump proper, or the passing from lower to upper stage, does not involve energy losses, except incidentally, and it is doubtful if the ordinary formula for loss by "sudden expansion" applies in this case.

3.—Mathematical Basis for Conclusions.

(*a*) *Relation Between Depth, Head, and Discharge.*—The mathematical deductions by which the foregoing conclusions were reached

are presented with the assistance of diagrams and as briefly as possible so as to avoid confusing mathematical details. The careful reader will notice that in some parts of the discussion the variation of velocity at different points in the cross-section of a channel and the variation of the slope of the hydraulic gradient under different conditions of flow, have been neglected. This has been done for the sake of simplicity, and does not affect the general conclusions as to the existence of the two alternative stages or the proposition that the hydraulic jump is not an absorber of energy by "sudden expansion". Referring to Fig. 3, the gradual slope of the hydraulic gradient represents channel friction losses.

H = the total head above the channel bed or distance up to the average hydraulic gradient for all the water in the stream;

d = depth of water;

V = average velocity;

Q = quantity of water discharged per unit width of channel.

This assumption, that we are dealing with a strip of unit width, is made for the sake of simplicity, and the conclusions will not be directly applicable to the full width of the stream unless the form of its cross-section approaches a rectangle.

From the relations, $d + \frac{V^2}{2g} = H$, and $V = \frac{Q}{d}$, we obtain the following general expression or equation giving the relation between depth, d, head, H, and discharge, Q:

$$d^3 - H d^2 + \frac{Q^2}{2g} = 0.$$

Note that this is a cubic equation in d, that is, for a constant head and discharge, there are three roots or three values of the depth, d. One of these comes out negative, leaving two real values corresponding to the two alternative stages, as shown in the following example.

Suppose the head, $H = 10$ ft., and the discharge, $Q = 50$ cu. ft. per sec. The general equation then becomes $d^3 - 10d^2 + 38.8 = 0$. Solving,* $d = 9.58$, 2.24, or -1.82. The last root, being negative, is

* The writer has never seen a formula arranged to give directly the three roots of any cubic equation. Hence the following is presented in the hope that others may find it useful:

If $x^3 + Ax^2 + Bx + C = 0$

$$x = \frac{2}{3}\left(+\sqrt{A^2 - 3B}\right)\cos.\frac{1}{3}\cos.^{-1}\frac{9AB - 2A^3 - 27C}{2\left(+\sqrt{A^2 - 3B}\right)^3} - \frac{A}{3}.$$

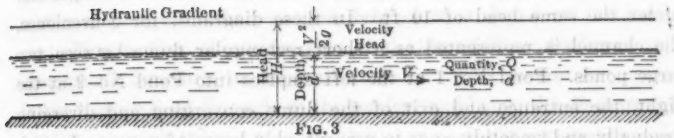


FIG. 3

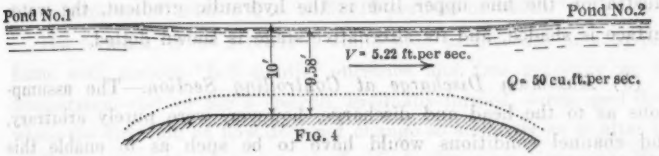


FIG. 4

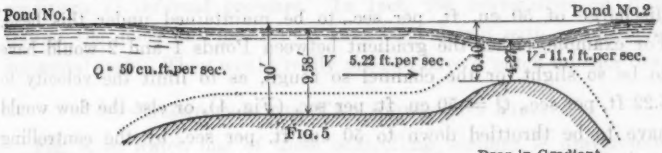


FIG. 5

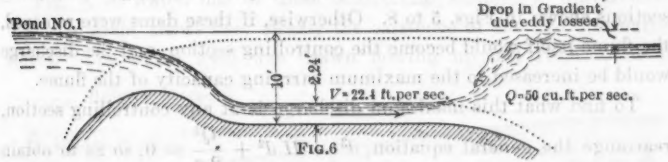


FIG. 6

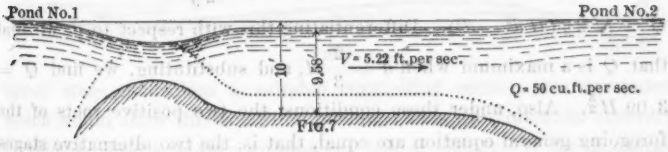


FIG. 7

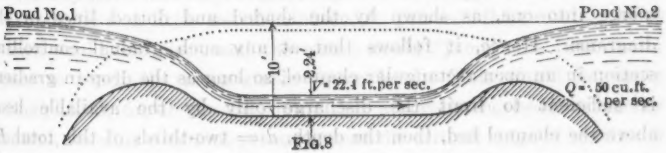


FIG. 8

unreal, and the two alternative stages correspond to depths of 9.58 and 2.24 ft., respectively. This condition is shown in Figs. 4 to 8, in which 50 cu. ft. per sec. are discharged at either of the two stages and under the same head of 10 ft. In these diagrams, for convenience, the channel is represented as a short rectangular flume between two large ponds. Pond No. 1 at the left empties into Pond No. 2 at the right, the entrance and exit of the flume converging and diverging gradually and smoothly so as to prevent eddy losses of energy. In each illustration the fine upper line is the hydraulic gradient, the water surface is shaded, and its alternative stage is shown dotted.

(*b*) *Maximum Discharge at Controlling Section.*—The assumptions as to the head and discharge, however, were purely arbitrary, and channel conditions would have to be such as to enable this discharge of 50 cu. ft. per sec. to be maintained under this head. For example, either the gradient between Ponds 1 and 2 would have to be so slight, or the channel so rough, as to limit the velocity to 5.22 ft. per sec., $Q = 50$ cu. ft. per sec. (Fig. 4), or else the flow would have to be throttled down to 50 cu. ft. per sec. by the controlling sections shown in Figs. 5 to 8. Otherwise, if these dams were removed, the flume itself would become the controlling section, and the discharge would be increased to the maximum carrying capacity of the flume.

To find what this maximum discharge is at any controlling section, rearrange the general equation, $d^3 - H\,d^2 + \frac{Q^2}{2\,g} = 0$, so as to obtain $Q = \sqrt{2\,g\,(H\,d^2 - d^3)}$. Differentiating this with respect to d, we find that Q is a maximum when $d = \frac{2}{3}\,H$, and substituting, we find $Q = 3.09\,H^{\frac{5}{2}}$. Also, under these conditions, the two positive roots of the foregoing general equation are equal, that is, the two alternative stages merge into one, as shown by the shaded and dotted lines in the diagrams. Hence, it follows that at any such gradual controlling section in an open rectangular channel, so long as the drop in gradient is sufficient to limit the discharge only by the available head above the channel bed, then the depth, $d =$ two-thirds of this total H, measured above the channel bed, and the discharge Q equals $3.09\,H^{\frac{5}{2}}$.

This is hardly to be taken as a practical formula for actual weir discharge, as the general equation for open-channel flow, on which it

is based, assumes that the pressure gradient coincides with the water surface, or that the water in every part of the section is under a pressure head equal to the vertical height of water above it. As a matter of fact, on a sharp-crested weir, or on a dam crest, or similar controlling section, the under side of the over-falling discharge is at atmospheric pressure, or approximately so. This means an increase in the velocity head throughout the section, and hence the discharge, Q, will exceed $3.09\ H^{\frac{3}{2}}$ per foot of width. Note that the Francis formula for a sharp-crested weir is $Q = 3.33\ H^{\frac{3}{2}}$ per foot of width.

The formula, $Q = 3.09\ H^{\frac{3}{2}}$, would hold in the case of a by-pass flume with smooth "bell-mouth" entrance and free get-away at the down-stream end, or for a smooth, gradually-formed, submerged dam, where the direction of flow does not change so suddenly as to alter the conditions of internal pressure. In fact, for convenience in illustrating, the dams in Figs. 5 to 8 have been drawn with a contracted horizontal scale. They would really need to have a much broader base.

4.—Examples of the Two Alternative Stages.

Fig. 9, in which one of these controlling sections is drawn more nearly to scale, shows very clearly the existence of the two alternative stages. The water level was drawn flowing up hill from lower to

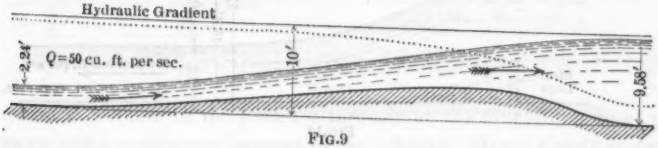


Fig. 9

upper stage, and then a suitable controlling section was computed which would allow this condition of flow. This illustrates the recovery of velocity head smoothly without any loss of energy. If the discharge was reduced to a little less than 50 cu. ft. per sec., the two alternative stages would not meet, and the water would "jump" the intervening space to reach the more stable upper level. This would be easily accomplished, for it can be shown that although the head, or energy content, remains constant, yet at or near the critical stage of maximum discharge, where the depth is equal to two-thirds of the head, the water surface is comparatively uncertain, and is easily subject to considerable fluctuation for very slight inequalities in effective area of

section. This tendency may be noted in a long flume under maximum discharge, such as a flood by-pass flume around a coffer-dam. It has this inherent tendency to surface fluctuations, which should not be aggravated by bends in the flume where it is possible to make the latter perfectly straight.

The existence of the two stages is also brought out in Figs. 6 and 7. Referring to Fig. 6: In case Pond No. 2, into which the flow empties, is low enough—so that, after the friction losses in the flume and all the eddy losses at the flume exit, there is still enough potential energy left to reach the pond level—then conditions will remain as in Fig. 6, with either a definite "hydraulic jump" as shown, or a series of more or less irregular waves, finally reaching the level of Pond No. 2. But if Pond No. 2 is higher than this, that is, so high that it cannot be reached by the total potential energy of the flume discharge, after the necessary friction losses of the high velocity in the flume and eddy losses caused by the jump have been deducted, then the pond will back up into the flume somewhat as shown in Fig. 7.

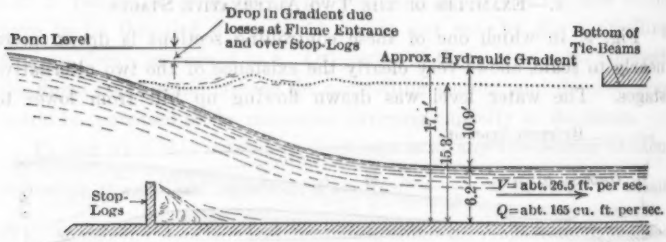


FIG. 10

This is apparently what happened in the case of a flume of the Massachusetts Metropolitan Water-Works, observed and reported by F. P. Stearns, Past-President, Am. Soc. C. E. Fig. 10 is taken from a print showing the measured velocities and surface profile in this flume, 40 ft. wide, emptying a reservoir at the left. The water is apparently flowing at the lower alternative stage, and the corresponding upper alternative stage is shown dotted. Besides what is shown in this figure the print shows the flume extending to the right a total length of 671 ft., with a bend in the middle which caused irregular waves in the down-stream half of the flume. Mr. Stearns states that during a later flood than the one shown here—possibly a greater flood, but only

slightly greater—he visited the flume and found the water at just about the level marked "bottom of tie-beams". Note the coincidence between the upper alternative stage and this point at which Mr. Stearns observed the water surface. Apparently, the discharge was not materially different in the two floods, but in the one case the particular channel conditions maintained the lower alternative stage and in the other the upper alternative stage.

5.—Examples of the Hydraulic Jump.

The gradual throttling of the flow represented in Figs. 5 to 8 permits the discharge to follow along either the lower or upper alternative stage, as shown particularly by comparing Figs. 6 and 7, but, in the case of the ordinary dam crest, the contraction is so sudden that the conditions of internal pressure are affected. The tendency is all

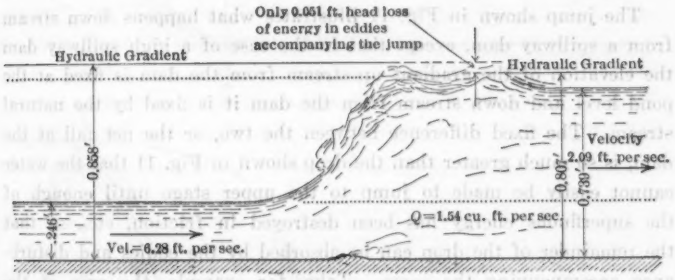


FIG. 11

toward a decrease in the pressure throughout the overfalling sheet of water, and a consequent increase in the velocity. Hence a weir or dam furnishes a velocity-creating condition, and tends to force the water level to drop to the lower or high-velocity stage, where it must continue until it encounters a velocity-destroying and pressure-creating condition, when it will rise again to the more stable upper stage of low velocity.

This rise is shown in Fig. 11, which represents one of four observations of the hydraulic jump by Bidone, quoted by Merriman as follows: The depth before the jump equals 0.246 ft.; the velocity before the jump equals 6.28 ft. per sec.; the depth after the jump equals 0.739 ft.; from these data the other quantities shown in Fig. 11 are easily computed closely enough for our purposes, assuming that the depths

before and after the jump were measured from the same floor line, parallel to the hydraulic gradient. An actual case similar to this, and one which brings out the importance of recognizing the two alternative stages, is illustrated in a recent issue of a technical journal.* A concrete chute, 9 ft. wide and 5 ft. high, at the end of the Truckee Main Canal was designed to discharge the canal flow at high velocity into the Lahontan Reservoir; but, when the water was turned in, it was found that with small discharges a vertical curve at the bottom of the chute was sufficient obstruction to cause the water to jump to the upper low-velocity stage, which was so high that it began to overtop the concrete sides and had to be retained by sand bags before the discharge increased sufficiently to maintain the lower stage throughout the length of the chute.

6.—Destructive High Velocity Below Spillway Dams.

The jump shown in Fig. 11 illustrates what happens down stream from a spillway dam, except that in the case of a high spillway dam the elevation of the gradient up stream from the dam is fixed at the pond level and down stream from the dam it is fixed by the natural stream. The fixed difference between the two, or the net fall at the dam, is so much greater than the drop shown in Fig. 11 that the water cannot easily be made to jump to the upper stage until enough of the superfluous energy has been destroyed in friction, etc., so that the remainder of the drop can be absorbed by the eddies and disturbance accompanying the jump. Take, for example, the case of the very high dam shown in Fig. 12. As in previous figures, the dotted line represents the stage alternative to the water surface. The friction under the excessive velocity causes the hydraulic gradient to slope down steeply, and the two alternative stages to approach each other until they are near enough for the jump to occur between them, with its accompanying eddies destroying the remainder of the energy, as already explained. The horizontal scale is contracted, due to the limits of the size of the drawing.

In Fig. 12 the profiles of the hydraulic gradient and alternative stages are more or less arbitrarily drawn, but they illustrate the great vertical distance between the two theoretic alternative stages, and the consequent tendency of the destructive high-velocity flow to continue

* *Engineering Record*, April 10th, 1915.

some little distance down stream from the dam. This is further illustrated in Fig. 1, showing the Granite Reef Dam of the Salt River project of the U. S. Reclamation Service.

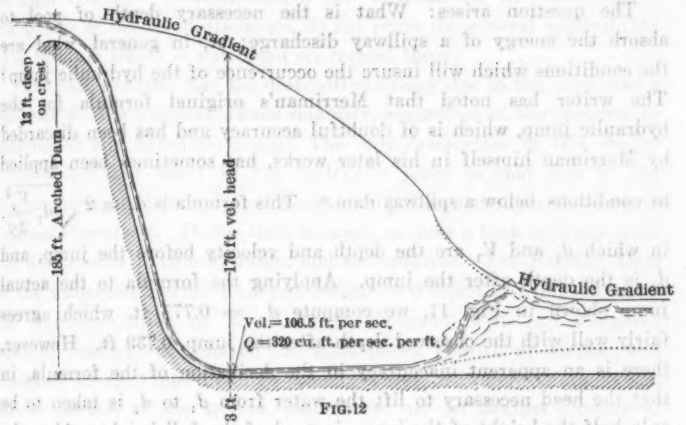


FIG. 12

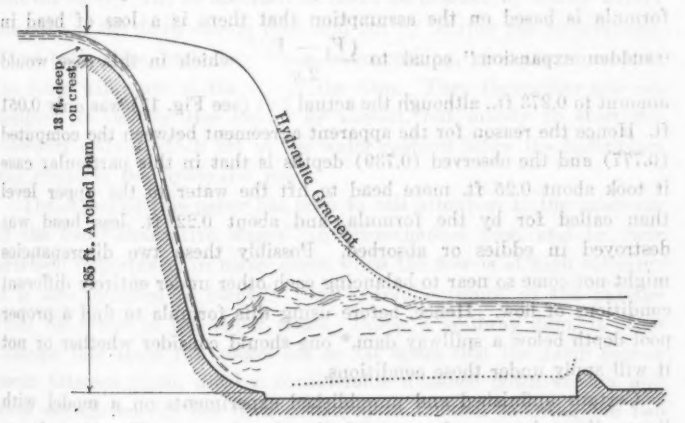


FIG. 13

Experience with actual spillways, as well as experiments on models, have shown that this tendency is counteracted, and the jump apparently brought back to the toe of the dam, if the pool into which the water plunges is deep enough. For example, as illustrated in Fig. 13, the

water-cushion is so deep, and the destruction of energy when the water strikes it is so great, that the total head is absorbed in this way, with no available energy left to maintain the high-velocity flow of Fig. 12.

The question arises: What is the necessary depth of pool to absorb the energy of a spillway discharge; or, in general, what are the conditions which will insure the occurrence of the hydraulic jump? The writer has noted that Merriman's original formula for the hydraulic jump, which is of doubtful accuracy and has been discarded by Merriman himself in his later works, has sometimes been applied to conditions below a spillway dam.* This formula is $d_2 = 2\sqrt{d_1 \frac{V_1^2}{2g}}$, in which d_1 and V_1 are the depth and velocity before the jump, and d_2 is the depth after the jump. Applying the formula to the actual jump shown in Fig. 11, we compute $d_2 = 0.777$ ft. which agrees fairly well with the observed depth after the jump, 0.739 ft. However, there is an apparent inaccuracy in the derivation of the formula, in that the head necessary to lift the water from d_1 to d_2 is taken to be only half the height of the jump, instead of the full height. Also, the formula is based on the assumption that there is a loss of head in "sudden expansion" equal to $\frac{(V_1 - V_2)^2}{2g}$, which in this case would amount to 0.273 ft., although the actual loss (see Fig. 11) was only 0.051 ft. Hence the reason for the apparent agreement between the computed (0.777) and the observed (0.739) depths is that in this particular case it took about 0.25 ft. more head to lift the water to the upper level than called for by the formula, and about 0.22 ft. less head was destroyed in eddies or absorbed. Possibly these two discrepancies might not come so near to balancing each other under entirely different conditions of flow. Hence, before using this formula to find a proper pool depth below a spillway dam,* one should consider whether or not it will apply under those conditions.

Certain unfinished and unpublished experiments on a model with linear dimensions one-forty-eighth the size shown in Fig. 13, indicate that, for these particular conditions also, at least in the model, the required amount of energy is destroyed and Merriman's original

* This use of the formula is not mentioned by Merriman, but has been made by other engineers. The latest edition of Merriman's "Hydraulics" treats the problem of the hydraulic jump differently.

formula gives approximately correct results, any pool depths greater than the computed 46 ft. causing the jump to occur at the toe of the dam and apparently preventing the water from shooting out straight at high velocity. Nevertheless, a few experiments not covering all conditions of head and discharge, do not prove the general applicability of a formula which is not based on correct theory.

The fact remains that the destructive energy due to the drop over a spillway dam is a definite, fixed quantity, regardless of the presence or absence of any hydraulic jump. The only destruction of this energy (conversion into heat) in the hydraulic jump is that due to the accompanying eddies and disturbances, and is measured by the drop in hydraulic gradient. If this drop is large, as over a high spillway dam, the disturbance is equally large, and unavoidable. The Bassano Dam of the Canadian Pacific Railroad Irrigation Project is equipped, after the recommendation of Mr. John R. Freeman, with two staggered rows of "baffle-piers", shaped like snow-plows, pointed up stream, and designed to split up the high-velocity sheet of water before it can strike the bed of the stream, and throw one jet against another so that the energy will be absorbed as much as possible by eddies within the body of the down-stream pool, and not by tearing the foundations. These baffle-piers, backed up by a water-cushion, give assurance that the jump will start at the toe of the dam. They themselves are not designed to destroy the energy by impact, but merely to start the necessary eddies, which act in the water-cushion below the baffle-piers and complete the hydraulic jump.

The object of this paper has been to call attention to the existence of the two "alternative stages" in open-channel flow, and to their practical importance, in many cases, when the flow is at high velocity; it is also intended to identify the "hydraulic jump" as the passage from the lower to the upper stage. Below a spillway dam, it often happens that these two stages are so far apart that the jump cannot occur between them, until a considerable distance down stream and after enough energy has been destroyed in friction to bring the two stages near together, as explained in the paper.

DISCUSSION

Mr. Groat. B. F. GROAT,* M. AM. SOC. C. E. (by letter).—The author has pointed out a principle which has commanded little or no attention. He has made clear the relations that control the limits of level within which the water in any channel must flow. Consequently, these relations control the upper limit of the possible rise of the hydraulic jump, although the effects of the channel and spillway conditions and, in particular, as Mr. Kennison indicates, the losses due to expanding cross-section, are still to be determined.

It appears that in any channel of usual shape there are two levels at which the water may flow with the same content of energy. Thus, so far as energy content is concerned, the water surface may change from one to the other of these levels without energy from any source other than that of the stream itself.

It also appears in the discussion, and this is clearly shown by Fig. 14, that the energy of the stream is always less for levels intermediate to the complementary levels, previously mentioned, than it is at either of them, and that it is greater outside of, than it is within, these limits. Therefore, the water cannot flow above the higher, or below the lower, of the complementary levels.

The great question is: What must happen at points between the two levels, at any of which there is a surplus of energy in the water? This surplus must appear in some form or another, the most likely being the energy which is absorbed by eddies. In other words, when the water surface is at either complementary level, does it not tend to move toward that intermediate level where the energy is a minimum? In the case of rectangular cross-sections, this level is at two-thirds the total energy head of the water; parabolic sections require three-fourths of the total energy head. At the foot of a dam the water may tend to rise to this level, but the rapid absorption of energy prevents it, as well as the fact that the shape of the channel below would not support such a flow in equilibrium, or there is no control section to support such a flow.

When the water surface is intermediate to the two complementary levels, it is certain that the formulas herein used do not apply, that is, the flow is not of the character supposed. In particular, the velocity is not uniform in the section, and there may be negative velocities, all of which destroy the value of the formulas for anything like accurate indications.

The height of the standing wave, for example, must depend on the shape, size, and character of the channel below the dam and also on the character of flow over the dam, as, for example, when changes of

* Pittsburgh, Pa.

FIG. 14.

Curve *A*.-Solution of the cubic equation $x^3-x^2+a=0$ for all values of a which furnish three real roots. This value of a is determined by the equation $a=-\frac{Q^2}{2\,g\,H^3}$ where Q is the discharge over one foot width of a dam and H the total hydrostatic head on the dam. A positive root of the equation represents the ratio of the theoretical depth below the dam to the total hydrostatic head, that is $x=d \div H$. The following limitations are noteworthy:

$0 < a < 4/27 = 2/3^3$

$-1/2 < x_1 < 0$

$0 < x_2 < 2/3$

$2/3 < x_3 < 1$

x_1, x_2, x_3 being the three roots of the equation, only one of which is negative for any admissible value of a.

Curve *B*.-Solution of the biquadratic equation $x^4-x^3+a=0$ for all values of a which furnish two real positive roots. These values of a are determined by the equation, $a=-\frac{Q^2}{2\,g\,k^2A^3}$, where $A=Kd^{3/2}$, A=area of cross-section of parabolic contour, d=the maximum depth above the bottom, $k=a$ constant, and H, as above, is the total hydrostatic head on the dam. A positive root of the equation is the ratio of the depth below the dam to the total hydrostatic head on the dam, or $x=d \div H$ as above.

The limitations are:

$0 < a < 27/256 = 3/4^4$

$0 < x_2 < 3/4$

$3/4 < x_3 < 1$

x_2 and x_3 being the two positive roots involved.

Q is the total discharge in the case of Curve *B*.

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condition of gate-opening occur in the case of arch dams provided with sluice-gates at the crest.

As the writer had the privilege of examining Mr. Kennison's treatment in connection with an important investigation, it may not be amiss to append some further study of the subject.

Fig. 14 solves problems relative to the two complementary levels for all channels having rectangular or parabolic cross-sections.

Using the author's notation, let:

$$a = \frac{Q^2}{2\,g\,H^3}, \text{ for rectangular sections;}$$

$$\text{and } a = \frac{Q^2}{2\,g\,K^2\,H^4}, \text{ for parabolic sections;}$$

where, in the latter case,

$$\text{the section area, } A = K\,x^{\frac{3}{2}}.$$

Let it be required, for example, to solve, by Fig. 14, the problem proposed by Mr. Kennison on page 344.

We have: $H = 10$ ft., $Q = 50$ cu. ft. per sec., and, therefore, $a = 50^2 \div 2\,g \times 10^3 = 0.0388$ for a rectangular section.

With this value of a, the corresponding values of the three ordinates to Curve A on Fig. 14, are 0.958, 0.224, and — 0.181.

As the curve relates to unit head, the three ordinates must be multiplied by H (10 ft. in this case), to secure the values of the corresponding depths. Therefore, the three possible depths for a total energy head of 10 ft. are 9.58 ft., 2.24 ft., and — 1.81 ft., of which the two positive values are the only ones sought. It will be seen that the results agree with the values deduced by Mr. Kennison by the solution of a cubic equation.

Curve B is used in exactly the same manner when the cross-section of the stream may be represented as the area of a parabola with its axis vertical and the vertex at the lowest point of the bed of the stream.

A better understanding of the subject and of the use of the curves will be gained by the following study of energy relations.

Rectangular Sections.—Let x be the depth in the channel, supposed to be rectangular, so that we need consider only 1 ft. of the width. The velocity is also supposed to be uniform, in order to simplify the discussion. Let v be the velocity and h the energy head. Then, the energy head at any point is given by

$$h = x + \frac{v^2}{2g} \quad \ldots\ldots(1)$$

If q is the discharge, we then have:

$$q = v\,x \quad \ldots\ldots(2)$$

or,

$$v = \frac{q}{x}$$

whence, $$x + \frac{q^2}{2\,g\,x^2} = h \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

or, $$x^3 - h\,x^2 + \frac{q^2}{2\,g} = 0.$$

If, in Equation (3), we put

$$a = q^2 \div 2\,g\,h^3 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

and $$r = x \div h$$

there results,

$$r^3 - r^2 + a = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

in which a is a variable depending on the value of x, that is, on r. In other words, the energy head depends on the depth of the water in the section and varies in a sense contrary to that of the variation of a. Thus, when a is a maximum, the energy head is a minimum, and *vice versa*, if we suppose the discharge constant. If h is supposed to be constant, then the discharge is a maximum simultaneously with a.

If the discharge is supposed to be constant, then it is clear that there are two levels, as Mr. Kennison has pointed out, either of which can be maintained with the same energy head, h, as it can be shown that two of the roots of the cubic equations, (3), or (5), are positive, and the third is negative.

Let it be required to determine the relation between the two positive roots of Equation (5) so that, if one of the two possible levels of the water is known, the other can be found.

In order to do this, let n be the numerical value of the negative root of Equation (5). Then, by the theory of equations:

$$\frac{r^3 - r^2 + a}{r + n} \text{ is an exact quotient} \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

that is,

$$r^2 - (n + 1)\,r + n\,(n + 1) = 0 \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

from which,

$$r = \frac{1 + n \pm \sqrt{(1 + n)\ \ (1 - 3\,n)}}{2} \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

This last equation determines the two positive roots in terms of the numerical value of the negative root. Table 1 has been computed by it, for the complete range of applicable values of r.

It will be seen from Equation (8) that n is limited to positive values which do not exceed $\frac{1}{3}$. At the limit 0, $r = 0$, or, $r = 1$, while the value, $\frac{1}{3}$, gives two equal values of r, that is, $\frac{2}{3}$ is a double root. This double root corresponds to a maximum value of a, and, consequently, to a minimum value of the energy head or to a maximum value of the discharge for a given energy head.

By Table 1, the graph of Equation (8) has been plotted as Curve *A*, on Fig. 14, which exhibits very clearly all the properties which have

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TABLE 1.—VALUES OF r, FOR THE EQUATION,

$$r^3 - r^2 + a = 0.$$

$n^3 + n^2 = a$	n	$\sqrt{(1+n)(1-3n)}$	r_1	r_2
0.000408	0.03	0.9792	0.9996	0.0204
0.001664	0.04	0.9567	0.9984	0.0417
0.003816	0.06	0.9323	0.9962	0.0639
0.006912	0.08	0.9060	0.9930	0.0870
0.011000	0.10	0.8775	0.9888	0.1113
0.016128	0.12	0.8466	0.9833	0.1367
0.02234	0.14	0.8131	0.9766	0.1635
0.02970	0.16	0.7767	0.9684	0.1917
0.03823	0.18	0.7367	0.9584	0.2217
0.04800	0.20	0.6928	0.9464	0.2536
0.05905	0.22	0.6441	0.9321	0.2880
0.07142	0.24	0.5892	0.9146	0.3254
0.08518	0.26	0.5265	0.8933	0.3668
0.10035	0.28	0.4525	0.8663	0.4138
0.11700	0.30	0.3606	0.8303	0.4697
0.13517	0.32	0.2298	0.7749	0.5451
$\frac{4}{27}$	$\frac{1}{3}$	0	$\frac{2}{3}$	$\frac{2}{3}$

been discussed and, in addition, that there is a double root, 0, which corresponds to a minimum value of a. This is also clearly shown by Equation (8).

An illustration of the use of the curve has already been given.

It is now desirable to express one of the positive roots of Equation (5) in terms of the other, so that, if the depth of water in a channel is known, the complementary depth for the equal energy head can be determined by a simple formula.

In order to deduce this formula, observe that, if d_1 and d_2 are the two complementary depths, one of which, say, d_2, is known, then the two corresponding complementary roots of Equation (7) are $r_1 = d_1 \div h$, and $r_2 = d_2 \div h$, and they furnish, therefore, the following two equations:

$$\left.\begin{aligned} r_1^2 - (n+1)\, r_1 + n\,(n+1) &= 0 \\ r_2^2 - (n+1)\, r_2 + n\,(n+1) &= 0 \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots (9)$$

As these equations must consist with each other, the relation between r_1 and r_2 may be deduced by eliminating n from them. This elimination may be effected most simply by taking the difference between the equations, thus:

$$r_1^2 - r_2^2 + (r_2 - r_1)\,(n+1) = 0 \ldots\ldots\ldots\ldots (10)$$

from which,

$$n + 1 = r_1 + r_2 \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

This relation might easily have been inferred from Equation (5), as the coefficient of r^2 is -1, that is,

$$n - r_1 - r_2 = -1 \ldots\ldots\ldots\ldots\ldots\ldots (12)$$

is the negative sum of the roots.

Substituting the values of n and $n + 1$ in either form of Equation (9), it is easy to show that

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$$r_1^2 + (r_2 - 1)\, r_1 + (r_2^2 - r_2) = 0 \quad \ldots\ldots\ldots\ldots (13)$$

which gives r_1 in terms of r_2 thus:

$$r_1 = \frac{(1 - r_2) + \sqrt{(1 - r_2)\ (1 + 3\, r_2)}}{2} \quad \ldots\ldots\ldots\ldots (14)$$

the negative radical being omitted from the numerator as both d_1 and d_2, and, therefore, both the roots must be positive.

It will be more convenient, however, to have d_1 and d_2 expressed directly as functions of each other. This may be accomplished by substituting for r_1 and r_2 in Equation (14), thus

$$\frac{d_1}{h} = \frac{1 - \frac{d_2}{h} + \sqrt{\left(1 - \frac{d_2}{h}\right)\left(1 + \frac{3\, d_2}{h}\right)}}{2} \quad \ldots\ldots\ldots (15)$$

from which,

$$d_1 = \frac{h - d_2 + \sqrt{(h - d_2)\ (h + 3\, d_2)}}{2} \quad \ldots\ldots\ldots\ldots (16)$$

and, by observing that

$$h = d_2 + \frac{v_2^2}{2\, g},$$

we may easily obtain,

$$d_1 = \frac{\frac{v_2^2}{2\, g} + \sqrt{\frac{v_2^2}{2\, g}\left(\frac{v_2^2}{2\, g} + 4\, d_2\right)}}{2} \quad \ldots\ldots\ldots\ldots (17)$$

or,

$$d_1 = \frac{v_2^2}{2\, g}\left(\frac{1}{2} + \sqrt{\frac{1}{4} + R}\right)$$

where,

$$R = \frac{d_2}{\frac{v_2^2}{2\, g}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (18)$$

R thus being the ratio of potential to velocity head.

Thus, d_2 being 2.24, and $v_2^2 \div 2\, g$ being 7.76, we should have $R = 2.24 \div 7.76 = 0.289$, from which,

$$d_1 = 7.76\ (0.5 + \sqrt{0.539}) = 9.58 \quad \ldots\ldots\ldots\ldots (19)$$

which agrees with Mr. Kennison's value on page 344. If we take $d_2 = 9.58$ and $v_2^2 \div 2\, g = 0.42$, in Equation (17), it will be found that $d_1 = 2.24$ and $v_1^2 \div 2\, g = 7.76$, a necessary result due to the reciprocal relation between d_1 and d_2. (See Equations (20).)

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Equation (17) may be deduced directly by solving

$$d_1 + \frac{v_1^2}{2g} = d_2 + \frac{v_2^2}{2g} \quad \ldots\ldots (20)$$

and,

$$d_1 v_1 = d_2 v_2$$

for d_1 in terms of d_2 after cancelling the factor, $d_1 - d_2$.

Now, it is easy to see that $0 < r_1 < \frac{2}{3}$ and $\frac{2}{3} < r_2 < 1$, and that either the equation, $r^3 - r^2 + a = 0$, or Fig. 14, showing the trace of the relation between a and r, shows that a is greater for all values of r between r_1 and r_2 than it is for r_1 or r_2; that is, h, the energy head of the flowing water, for all depths between d_1 and d_2, is less than it is at either of these limits, and the energy is greater at depths below d_1 or above d_2. Hence, the flow of the water is limited to some depth, x, where $d_1 < x < d_2$. That is, the water may flow at any level between d_1 and d_2, but not outside these limits. This, of course, supposes a rectangular channel and that a sufficient amount of energy is absorbed where the level is at neither limit.

Parabolic Cross-Sections.—The foregoing method of analysis may be extended to shapes of section other than rectangular with corresponding modifications. Suppose, for example, that the section area is connected to the depth by

$$A = k x^{\frac{3}{2}} \quad \ldots\ldots (21)$$

A being the area, x the depth, as before, and k a constant.

Then,

$$h = x + \frac{v^2}{2g} \quad \ldots\ldots (22)$$

$$q = A v. \quad \ldots\ldots (23)$$

and,

$$v = \frac{q}{A} = \frac{q}{k x^{\frac{3}{2}}} \quad \ldots\ldots (24)$$

Therefore,

$$h = x + \frac{q^2}{2g k^2 x^3} \quad \ldots\ldots (25)$$

$$x^4 - h x^3 + \frac{q^2}{2g k^2} = 0 \quad \ldots\ldots (26)$$

from which,

$$r^4 - r^3 + \frac{q^2}{2g k^2 h^4} = 0$$

or,

$$r^4 - r^3 + a = 0 \quad \ldots\ldots (27)$$

where,

$$a = \frac{q^2}{2g k^2 h^4} \quad \ldots\ldots (28)$$

and,

$$r = x \div h.$$

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It is then easy to show, by reasoning exactly as in the case of a rectangular channel, that a is a maximum and H a minimum when $r = \frac{3}{4}$, that is, when the depth is three-fourths of the total energy head. The graph of the equation for the applicable values of the roots may be constructed from Table 2.

TABLE 2.—VALUES OF r FOR THE EQUATION,

$$r^4 - r^3 + a = 0.$$

r	r^3	r^4	a	Remarks.
0	0	0	0	
0.1	0.0010	0.0001	0.0009	
0.2	0.0080	0.0016	0.0064	
0.3	0.0270	0.0081	0.0189	
0.4	0.0640	0.0256	0.0384	
0.5	0.1250	0.0625	0.0625	
0.6	0.2160	0.1296	0.0864	
0.7	0.3430	0.2401	0.1029	
¾	0.4219	0.3164	0.1055*	$(\frac{3}{4})^3(1-\frac{3}{4}) =$ maximum value of $a = \frac{3^3}{4^4}$.
0.8	0.5120	0.4096	0.1024	
0.9	0.7290	0.6561	0.0729	
1.0	1	1	0	

The curve is marked B on Fig. 14. Only two positive roots are applicable, there being a pair of imaginary roots for the admissible range in value of a, the maximum of which is $3^3 \div 4^4 = 0.1055$, when $r = \frac{3}{4}$.

Let it be required, as in the former case, to deduce the relation between the two positive roots, and thus a formula for determining the second possible water level when the first is known, supposing that no losses of energy occur in the change.

This may be accomplished by a procedure analogous to that adopted in the case of rectangular cross-sections, but the following will probably be found more expeditious:

Let d_1 and d_2 be the two possible levels. We then have,

$$d_1 + \frac{v_1^2}{2g} = d_2 + \frac{v_2^2}{2g} \quad \ldots\ldots(29)$$

but,
$$v^2 = q^2 \div A^2 = q^2 \div K^2 x^3 \quad \ldots\ldots(30)$$

therefore,
$$d_1 - d_2 = \frac{q^2}{2gK^2}\left(\frac{1}{d_2^3} - \frac{1}{d_1^3}\right)$$
$$= \frac{q^2}{2gK^2}\,\frac{d_1^3 - d_2^3}{d_1^3 d_2^3} \quad \ldots\ldots(31)$$

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from which we easily find

$$1 = \frac{q^2}{2\,g\,K^2\,h^4}\,\frac{d_1}{r_1^4}\,\frac{d_1^2 + d_2\,d_1 + d_2^2}{d_2^3} \quad \ldots\ldots\ldots\ldots\ldots\ldots (32)$$

Thus, by Equation (28),

$$1 = \frac{a}{r_1^4}\,(R^3 + R^2 + R) \quad \ldots\ldots\ldots\ldots\ldots\ldots (33)$$

or,

$$R^3 + R^2 + R - b = 0,$$

where R is the ratio of the two applicable roots, and,

$$b = \frac{r_1^4}{a} = \frac{d_1}{\frac{v_1^2}{2\,g}} = \frac{r_1}{1 - r_1} \quad \ldots\ldots\ldots\ldots\ldots\ldots (34)$$

It may be shown that two of the roots are imaginary for all applicable values of b, that is, of a and r_1, so that there will be only one solution, which is given by,

$$A = \frac{7}{54} + \frac{b}{2}$$

$$B = \sqrt{A^2 + \left(\frac{2}{9}\right)^3}$$

$$R = (A + B)^{\frac{1}{3}} + (A - B)^{\frac{1}{3}} - \frac{1}{3} \quad \ldots\ldots\ldots\ldots\ldots\ldots (35)$$

For example, if $r_1 = 0.45$, then $b = 45 \div 55 = 9 \div 11 = 0.8181$ and $a = 0.050119$, from which,

$$A = \frac{7}{54} + \frac{b}{2} = 0.538720$$

$$B = \sqrt{A^2 + \left(\frac{2}{9}\right)^3} = 0.548811$$

$$A + B = 1.087531$$

$$A - B = -\,0.010091$$

and

$$R = 1.087531^{\frac{1}{3}} - 0.010091^{\frac{1}{3}} - \frac{1}{3}$$

$$= 1.028365 - 0.216094 - \frac{1}{3}$$

$$= 0.47894.$$

Therefore, we must have,

$$r_2 = \frac{r_1}{R} = \frac{0.45}{0.47894} = 0.93957,$$

the correctness of which may be verified by reference to Fig. 14, for $a = 0.0501$. It will be seen that the two values of r, for this value of a, agree with the values just computed.

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It is now easy to see how any shape of cross-section can be treated in a manner similar to the foregoing, whether the section is, or is not, expressible in algebraic form. It is also clear, from the foregoing treatment, that the complementary levels vary relatively for different shapes of section, and that the elevation of the water surface in control sections also depends on the shape as well as on the magnitude of the sections, being two-thirds of the total energy head for rectangular sections and three-quarters of the energy head for parabolic sections. The jump, likewise, is governed by the shape of the channel.

An interesting question arises in the case of cross-sections other than rectangular. If individual widths are treated separately, it will be found that the corresponding complementary levels vary according to location across the stream. That is, theoretically, the water surface should present a curved transverse profile. May this not be one of the prime causes of much of the disturbance which occurs in streams of irregular cross-section?

The writer is of the opinion that we wander too far from the domain of theoretical hydrodynamics when we apply equilibrium equations to finite masses and volumes of water. This can be done with exactness only when we are considering rigid bodies. The difficulty has always been that of integrating the differential equations throughout the volumes and over the bounding surfaces of the masses of water involved. However, much progress has been made and still more remains for the future. In time, we shall understand the nature of the losses which occur by reason of expanding sections in draft-tubes and in the hydraulic jump. Mathematics and experimental physics must join hands in the work. Fortunately, the density is practically constant, which simplifies matters considerably, but our knowledge of the effects of viscosity is very limited.

Varied examples of the hydraulic jump may be seen at most sea beaches where the slope is not too steep. A favorable condition occurs when the slope is moderate for a considerable distance up the beach followed by a steeper slope above. The steeper slope aids in accelerating the water receding down the beach. When the receding water attains the right velocity with respect to an incoming wave, the jump occurs. These jumps seldom exceed 1 ft. in height, but present an interesting variety of phenomena.

Mr. Muckleston.

H. B. MUCKLESTON,* M. AM. SOC. C. E. (by letter).—This interesting paper should be very valuable to those who have to deal with the phenomena discussed. Two instances are cited where the question proved important for widely different reasons, and many others will readily occur to the reader.

It will be understood that the formulas presented are like all other hydraulic formulas founded on incomplete theory, or probably—put in

* Calgary, Alberta, Canada.

Mr. Muckleston.

a better way—on theory founded on incomplete or erroneous assumptions, and that in practice they will have to be modified by a coefficient.

The author points out one case, in the application to weir discharge, where the theoretical formula must be modified to agree with the results of experiment.

On page 344 the author makes the convenient but inaccurate assumption that the velocity head of a flowing stream is equal to $\frac{V^2}{2g}$, where V is equal to the discharge divided by the cross-sectional area. This is never quite true, and in extreme cases may be very far from the truth. Assume, for example, a stream divided into two parts, and that the mean velocity is 14 ft. per sec. in the upper part and 2 ft. per sec. in the lower. On the author's assumption, the velocity head for the whole would be 1.0 ft., nearly. Taking the two values separately, and using the mean of the values thus found, it would be about 1.56 ft.

The case is extreme, and probably impossible, but it serves to show that the assumption is not correct.

Referring now to the case of the Bassano Dam, cited by the author and illustrated by Fig. 2: This dam is about 38 ft. high above the down-stream apron, and may discharge about 13 ft. deep over the crest. The concrete apron below the ogee is about 50 ft. long, and it was feared that at certain stages the jump or standing wave might be pushed down stream to such a distance that it would occur on the unprotected river bed, which in this case is gravel overlying clay. In order to insure that this should not occur, on the recommendation of Mr. John R. Freeman, two rows of baffle piers were built on the ogee, as described by the author. The dam has now been through two seasons and one very large flood, and it may be interesting to give the results of close observation of the behavior of these baffles.

Owing to the presence of two large under-sluices near the center of the dam at apron level, a length of 60 ft. was not equipped with them, and another length next to the east abutment was left unprovided on account of a turbine outlet. There was thus an opportunity to observe closely, especially at moderate stages, the results of building these baffles. It should be noticed that the level of the stream below the dam rises very much faster than does the pond above, and that, in consequence, the depth of cushion increases very much faster than the discharge. For instance, with about 4 ft. on the crest, the depth on the apron is about the same; for 10 ft. on the crest, the depth on the apron was about 18 ft.

It was noticed that at very low stages, say, less than 3 ft., the stream striking the baffles was so thin that practically all the energy was absorbed by impact. As the stage increased, the effect of the baffles was seen in a series of jets which did not impinge on one an-

FIG. 15.—BASSANO DAM: OVERFLOW WITH 5.5 FT. ON THE CREST.

FIG. 16.—BASSANO DAM: OVERFLOW WITH 9.5 FT. ON THE CREST.

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Mr. Muckleston.

other, but, on the contrary, were thrown out on the apron, especially the jets from the upper row. This effect increased in volume up to a stage of about 6.5 ft., when the depth on the carpet began to mask the baffles, and the violence of the jets began to decrease, until at a stage of about 8.5 ft. the action of the baffles was completely obscured by the water-cushion.

Considering that part where no baffles were provided, it was observed that for stages up to about 3 ft. the jump occurred on the ogee. (The stage varies because the depth of cushion for any given stage varies somewhat on account of the condition of the stream bed below.) From this up to about 7 ft. the jump is pushed down stream for a short distance, estimated at about 20 ft., after which it begins to return. Figs. 15 and 16, reproduced from photographs taken from nearly the same point, illustrate the action at two different stages. Fig. 15 shows the dam at about 5.5 ft. on the crest. The unequipped length is very evident, as is also the action of the baffles in throwing out the almost horizontal jets. Fig. 16, showing the dam with 9.5 ft. on the crest, is specially interesting, when compared with Fig. 15. The section of the dam not equipped is not at all evident, and cannot be located except by counting the bridge piers. Looking down from the bridge, there did not appear to be any difference in the action of the two parts.

The author's conclusions are interesting and valuable, but there is a further conclusion which he did not draw. The writer is of the opinion that the ogee is the wrong method of treating a spillway dam, and that any device which may be added to minimize its bad effects is only a palliative and not a cure. A straight overfall into a deep water-cushion is Nature's method, and, if adopted in the work of Man, no evil effects from jump, back roll, standing wave, or whatever it may be called, need be feared. As far as the writer knows, this has never been investigated, except as regards the effects on the floor of the cushion. Many years ago, Col. Dyas experimented in India by sending thin glass bottles over falls, and observed that they were not broken when the depth of the cushion exceeded $h^{\frac{1}{2}} d^{\frac{1}{3}}$, where h is the height of the fall and d is the depth on the crest. For design, this was modified to one-half or one-third of the depth thus found, depending on the material of the floor.

Mr. Stearns.

FREDERIC P. STEARNS,* PAST-PRESIDENT, AM. SOC. C. E. (by letter).—This paper is a very valuable contribution on a subject which has rarely been discussed, but is of high practical importance. The treatment of the subject by the author from a theoretical and mathematical standpoint is so complete that the writer can add nothing of value in these respects.

* Boston, Mass.

Mr. Stearns.

The author makes this statement:

"A knowledge of the hydraulic principles involved [in the hydraulic jump] should enable destructive high velocities and turbulence to be avoided, or intelligently provided for, in the design of flumes, dam foundations, etc."

Elsewhere in the paper, also, he calls attention to the practical dangers of permitting water to flow with high velocity at the lower "alternative stage". It seems desirable that the dangers involved in such high velocities and the methods by which they may be prevented should be still further emphasized.

In the writer's experience, damage occurred to a wooden flume, referred to in the paper, because the water flowed at the lower "alternative stage" for a long distance before the hydraulic jump occurred. The flume, 40 ft. wide, 16 ft. high at its upper end, and 13 ft. at the lower end, was built to carry the water of a stream past a masonry dam during the construction of its lower portion. It was built of planed lumber, with no obstructions on the inside. Its total length was about 700 ft. At a point 380 ft. from its upper end there was an angle in the flume, the deflection from a straight line being 8.5 degrees. The approach was flared, to prevent losses of head due to contraction at the entrance.

The upper end of the flume passed through an earth coffer-dam, and, on the center line of the latter, 30 ft. below the entrance to the flume, provision was made for removable stop-logs, which were kept in place except during floods to maintain a water supply through an aqueduct. The plans provided for a coffer-dam across the valley at the lower end of the flume, but at the time of the floods referred to subsequently this dam had not been built.

In designing the flume, the maximum quantity of water to be discharged during the greatest floods was assumed to be 9 000 cu. ft. per sec., and it was expected that with this discharge there would be a loss of head at the entrance of about 5 ft., and that the flume would run nearly full.

The writer recognized at that time that immediately below where the drop in the surface occurred, at the entrance to the flume, the water surface might be lower than at points farther down stream, but there was nothing in his experience or that he had gleaned from the experience of others to lead him to think that the low-water surface would persist for any long distance.

A flood substantially equal to that assumed in designing the flume occurred, and the flume operated substantially in accordance with the expectations at the time it was designed, as there was a moderate loss of head at the entrance and the flume ran nearly full.

Soon afterward, a second flood occurred, somewhat smaller than the first, but the water, instead of nearly filling the flume, ran with high

velocity with a depth of less than half the height of the flume, until it reached the slight angle 380 ft. from its upper end. The hydraulic jump then took place, not all at once, but rather gradually, in the next 275 ft. The water on the outside of the flume, corresponding as it did to the height of water at its lower end, was from 1 to 2 ft. higher than the water inside, where it was flowing at the low stage, thereby causing an upward pressure on the bottom sufficient to lift the flume off its foundation for a considerable part of its length, notwithstanding the fact that it was anchored in some places to the ledge and in other places was fastened to anchors buried 3 ft. below the surface of the ground which supported the sills. Mr. Stearns.

Just why the water flowed at such different depths in the flume in the two floods is not definitely known, but it is probable that it was due to the removal of all the stop-logs and other obstructions at the time of the first flood and to leaving a part of these logs in place during the second flood. The author illustrates the conditions toward the upper end of this flume in Fig. 10.

The damage done to the flume might have been avoided had some comparatively small obstructions been placed in its bottom toward the upper end, so as to cause the hydraulic jump to take place there. Very few flumes are built where the damage which occurred in this case would be likely to occur, and the incident has been fully described in order to illustrate a cause of danger which is frequently present below spillway dams.

Taking now the case of a high spillway dam, like that illustrated by Fig. 12, the water attains a high velocity as the result of the fall from the reservoir, and, owing to the easy curve at the bottom of the dam, is projected horizontally along an apron, with the velocity still high, and a correspondingly small depth, but at a greater or less distance below the dam the hydraulic jump occurs, and down stream from this jump the water is much higher than above it.

Assuming, in a case like that described and illustrated, that there is a level concrete apron without vents to relieve the pressure beneath it, over which the water moves with high velocity, and that the hydraulic jump takes place at the lower edge of the apron, it will then seem obvious that the water pressure beneath the apron will be that due to the comparatively high level of the water down stream from the hydraulic jump, although this may be somewhat increased by the pressure transmitted from the reservoir itself through the strata beneath the dam. In other words, the pressure tending to lift the concrete apron will not be less than that represented by the difference of level between the surface of the swiftly moving water up stream from the jump and the slower moving water down stream from it. If the excess of pressure beneath the apron is greater than the weight of the apron, it will lift.

Mr. Stearns.

The author's Fig. 12, based on a spillway dam 185 ft. high, is, so he states, "more or less arbitrarily drawn", but it is sufficiently near what may occur in practice to be used for the purpose of illustration. By scaling from the illustration, the jump from the low stage to the high one is 34 ft., and if the level portion below the dam was a concrete apron without vents, a thickness of fully 23 ft. would be required to balance this net head, assuming the concrete when immersed in water to weigh 90 lb. per cu. ft.

It is frequently the case that there is trouble below spillway dams as the result of heavy floods, and an important factor in causing such trouble is a failure to stop the high velocity close to the foot of the dam.

High velocity below a dam may cause trouble in two ways:

(*a*)—In the manner already described, owing to the small pressure of the swiftly moving water on the top of a nearly level apron, or on the surface of a ledge, in comparison with the pressure under the apron, or in the seams of the ledge when these communicate with the slowly moving water at a higher level below the jump;

(*b*)—By reason of the swiftly moving water impinging upon a crack or seam facing up stream, thereby causing in the crack or seam the increased pressure due to the velocity of the water, much in the same way that increased pressure or head is produced in a Pitot tube used in a swiftly moving stream when the orifice faces the current.

From one or the other or both of these causes, aprons may be lifted, great masses of rock lying in horizontal strata may be displaced, and holes may be dug in the rock below important spillway dams to an extent that seems nearly incredible to those who have not witnessed the results or studied the forces acting to produce such movements.

The author makes this statement, on page 353:

"The fact remains that the destructive energy due to the drop over a spillway dam is a definite, fixed quantity, regardless of the presence or absence of any hydraulic jump."

The writer agrees with this statement, if the word "destructive" is omitted.

There is a definite amount of energy due to the fall of the water, but it may or may not be destructive. Water may fall through a turbine, converting the energy due to the drop into other forms of energy, and yet be in no wise destructive. Similarly, the energy due to the drop over a spillway dam may be converted into heat close to the foot of the dam, without being destructive, if the material upon which the water drops can be depended on to endure the shock and wear caused by the swiftly moving water and by all other substances which are carried by it over the dam.

The common practice of curving the bottom of a concrete dam, so as to throw the sheet of water out in a nearly horizontal direction with

Mr. Stearns.

little loss of velocity, is one which seems to have had its origin either in the idea that this changing of the direction of the water would prevent erosion of the material below the dam, or that the material used in constructing the dam could not be depended on to stand the shock of the falling water and other substances. The use of this bottom curve has signally failed to prevent the erosion of rock and other material below a dam, and materials are now available for constructing dams, which under most circumstances can be depended on to endure the shock and wear resulting from the drop of the water in great quantities over a very high dam.

The writer believes that the policy in respect to the lower portion of spillway dams should be modified, and that the high velocity should be destroyed at the point where the water reaches the bottom of the dam, and not be merely changed in direction.

The high-grade concrete which it is feasible to make at the present day, if used in sufficient thicknesses and amply reinforced with steel, can be depended on in nearly all cases to endure the shock and wear caused by water and all other substances which fall over a dam, and, if necessary, points subject to excessive shock and wear can be faced with cast iron. The shock can be greatly lessened, where it is feasible to provide an adequate water-cushion.

The method to be used in different cases will naturally vary. On page 353 the author has referred to one method:

"The Bassano Dam of the Canadian Pacific Railroad Irrigation Project is equipped, after the recommendation of Mr. John R. Freeman, with two staggered rows of 'baffle-piers', shaped like snow-plows, pointed up stream, and designed to split up the high-velocity sheet of water before it can strike the bed of the stream, and throw one jet against another so that the energy will be absorbed as much as possible by eddies within the body of the down-stream pool, and not by tearing the foundations. These baffle-piers, backed up by a water-cushion, give assurance that the jump will start at the toe of the dam."

Fig. 2 is a view of this dam in action.

The writer, as already indicated, believes that, as a rule, a more complete destruction of the high velocity at the foot of the dam should be attempted. One method of doing this is indicated by Fig. 17, which represents (with slight modification) the arrangements provided at a spillway dam designed in part by the writer.

It was necessary in this case, and it is frequently necessary in order to obtain a suitable foundation for a dam, to excavate to a considerable depth below the river bed, and if the down-stream face of the dam is continued to below the bed without a curve, and the excavation on the down-stream side is filled with concrete to a level below the bed, a large thickness of concrete upon which a sheet of water may strike is easily provided; also a water-cushion.

Mr. Stearns.

The sketch, Fig. 17, illustrates this method of construction. The concrete on the down-stream side is built to a level about 5 ft. below the river bed, and 15 ft. from the down-stream face a wall is built to the level of the river bed. By such an arrangement, there is a pool having a depth of 5 ft. in addition to the depth to which the water rises above the river bed with different river discharges. If, in the case of a big flood, the water in the river rises 10 ft., the sheet will fall into a pool 15 ft. deep.

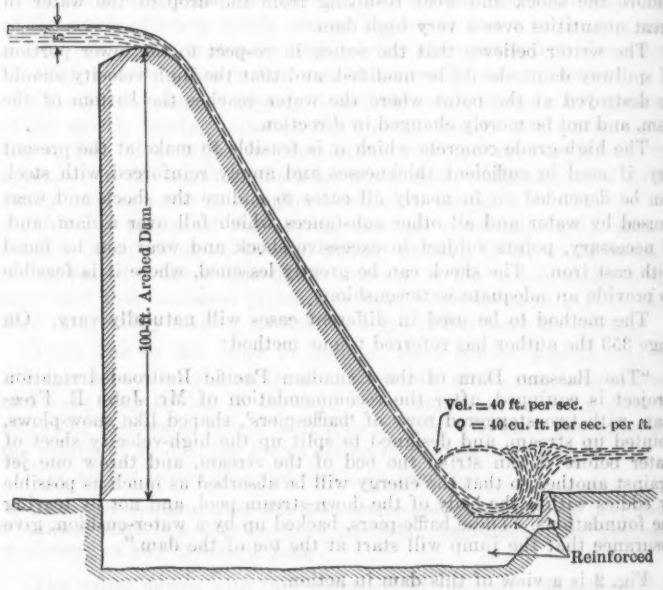

FIG. 17.

In order to visualize the conditions existing at such a dam during a maximum flood, let it be assumed that the maximum depth on the crest will be substantially 5 ft. and the quantity of water per linear foot 40 cu. ft. per sec. The velocity of the descending water, where it enters the pool at the foot of the dam, will be between 30 and 60 ft. per sec., making the depth of the sheet at this place between 16 and 8 in.

To use an intermediate figure between these limits, it may be assumed for the purpose of discussion that the sheet will be 1 ft. thick and have a velocity of 40 ft. per sec. Water with this velocity entering the pool at the face of the dam would obviously continue to flow down

the face to the bottom of the pool. It would entrain some of the water through which it passed, thereby decreasing the velocity and increasing the thickness of the moving mass. On reaching the bottom, the water would be deflected down stream, and would flow with still diminishing velocity, for the most part horizontally, until its motion was retarded by the wall 15 ft. from the dam; thence it would flow upward, most of the water turning down stream, but a part of it turning up stream to take the place of the water entrained. Mr. Stearns.

It seems quite clear that with the proportions thus given for the depth of the pool and the thickness of the flowing sheet, the energy due to the drop of 100 ft. would be thoroughly taken care of in the pool within 15 or 20 ft. of the bottom of the down-stream slope, without destructive action, and that at points farther down stream there would be no high velocity due to the fall of the water over the dam.

If there was any doubt as to the action thus described taking place in a pool at the base of a dam, the further precaution might be taken of building occasional projections from the face of the dam, irregularly spaced at different distances below the flood level in the river. Such projections should be strong enough to resist all shocks, and would deflect parts of the sheet of water, making it impossible for it to flow at the lower "alternative stage". The confused currents caused by such projections would quickly entrain large quantities of the water in a pool, thereby producing a fairly regular flow within a very short distance from the foot of the dam.

The design of the spillway at the Gatun Dam at the Panama Canal contains features to be commended, especially in cases where the quantity of water per linear foot of spillway is very large. In this case, the raising of large gates on the spillway crest, 18 ft. lower than the reservoir at its maximum level, causes great volumes of water per linear foot to flow over the dam. The drop from the crest to the concrete apron at the foot of the dam is 59 ft., and the thickness of the apron where the water falls upon it is about 12 ft. Projecting above the apron, close to the foot of the dam, there are two rows of baffles with vertical up-stream faces, those in the upper row being about 18 ft. wide and 8 ft. high. The baffles in the two rows are staggered, so that the water passing between those in the upper row impinges upon those of the lower row. When in action, much of the water is thrown high into the air, with a nearly complete destruction of its horizontal velocity, and cross-currents are also formed. The total result is that most of the energy of the water is taken care of at the baffles, and beyond them the water flows away with a comparatively low velocity. The baffles are faced on the up-stream side with heavy cast iron anchored to them, and these in turn are securely anchored by reinforcing rods to the mass of concrete beneath.

Mr. Stearns.

It seems unnecessary to multiply illustrations. There are many expedients by which the high velocity of the water falling over a dam can be checked very close to its foot, and the dangerous features of the high velocity and the hydraulic jump farther down stream can be avoided.

Mr. Andrews.

CARL B. ANDREWS,* ASSOC. M. AM. SOC. C. E. (by letter).—In the author's discussion of the depth of pool necessary to absorb the energy of a spillway discharge, he mentions Professor Merriman's original formula for height of jump. It would perhaps be only fair to Professor Merriman to quote the paragraph in connection with the development of this formula. It is as follows:

"To determine the height of the jump, let $d_2 - d_1$ be represented by j. It is then to be observed that the lost velocity-head is $(v_1^2 - v_2^2) \div 2\,g$, and that this is lost in two ways, first by the impact due to the expansion of section $\left[h' = \frac{(v_1 - v_2)^2}{2\,g}\right]$, and second by the uplifting of the whole quantity of water through the height, $\frac{1}{2}(d_2 - d_1)$, loss in friction between d_1 and d_2 being neglected.

Hence $$\frac{v_1^2 - v_2^2}{2\,g} = \frac{(v_1 - v_2)^2}{2\,g} + \frac{j}{2}.$$

Inserting in this the value of v_2, found from the relation $v_2\,(d_1 + j) = v_1\,d_1$, and solving for j, gives

$$j = -\,d_1 + 2\sqrt{d_1 \frac{v_1^2}{2\,g}}."$$

The subject of the height of jump has been treated by at least two other eminent engineers, Professor Hubert Engels, and Professor W. C. Unwin. Professor Engels, in his recently published "Handbuch des Wasserbaues," deduces the approximate height of the jump, as follows: The height of the jump may be obtained approximately with the aid of the law of kinetic energy, by equating the difference between the two velocity heads to the difference between the two water surfaces, or (in Fig. 18),

$$\frac{v_2^2}{2\,g} - \frac{v_1^2}{2\,g} = d_1 - d_2$$

or, since $$v_2 = v_1 \frac{d_1}{d_2},$$

and letting $$\frac{v_1}{2\,g} = k.$$

$$\frac{1}{2\,g}\left[v_1^2 \frac{d_1^2}{d_2^2} - v_1^2\right] = \frac{v_1^2}{2\,g}\left[\frac{d_1^2 - d_2^2}{d_2^2}\right] = d_1 - d_2$$

* Honolulu, Hawaii.

or $$\frac{v_1^2}{2g}\left[\frac{d_1 + d_2}{d_2^2}\right] = 1$$

or $$k\left[\frac{d_1 + d_2}{d_2^2}\right] = 1$$

whence, $$d_2^2 - k\,d_2 = k\,d_1$$

$$d_2 = \frac{k}{2} + \sqrt{k\,d_1 + \frac{k^2}{4}} = \frac{v_1^2}{4g} + \sqrt{\frac{v_1^2}{2g}d_1 + \left(\frac{v_1^2}{4g}\right)^2}$$

or $$j = -\,d_1 + \frac{v_1^2}{4g} + \sqrt{\frac{v_1^2}{2g}d_1 + \left(\frac{v_1^2}{4g}\right)^2}.$$

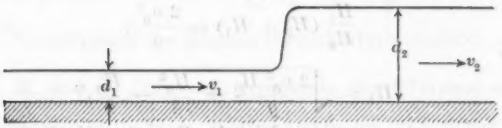

FIG. 18.

Professor Unwin's treatment of the problem appears in the *Encyclopedia Britannica*, under the head of "Hydraulics". He writes:

"Let the figure [Fig. 19] represent the longitudinal section of a stream and *ab*, *cd*, cross-sections normal to the bed, which for the short distance considered may be assumed horizontal. Suppose the mass of water *abcd* to come to $a'b'c'd'$ in a short time t; and let u_0, u_1, be the velocities at *ab* and *cd*, Ω_0 and Ω_1 the areas of the cross-sections. The force causing change of momentum in the mass *abcd* estimated horizontally is simply the difference of the pressures on *ab* and *cd*. Putting h_0, h_1, for the depths of the centers of gravity of *ab* and *cd* measured down from the free water surface, the force is $G\,(h_0\,\Omega_0 - h_1\,\Omega_1)$ pounds, [G = weight per cubic unit of liquid] and the impulse in t seconds is $G\,(h_0\,\Omega_0 - h_1\,\Omega_1)\,t$ second-pounds. The horizontal change of momentum is the difference of the momenta of $cdc'd'$ and $aba'b'$; that is,

$$\frac{G}{g}\,(\Omega_1\,u_1^2 - \Omega_0\,u_0^2)\,t.$$

Hence, equating impulse and change of momentum,

$$G\,(h_0\,\Omega_0 - h_1\,\Omega_1)\,t = \frac{G}{g}\,(\Omega_1\,u_1^2 - \Omega_0\,u_0^2)\,;$$

Hence $$h_0\,\Omega_0 - h_1\,\Omega_1 = \frac{1}{g}\,(\Omega_1\,u_1^2 - \Omega_0\,u_0^2).$$

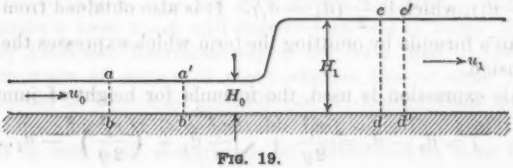

FIG. 19.

Mr. Andrews.

"For simplicity let the section be rectangular, of breadth B and depths H_0 and H_1 at the two cross-sections considered; then $h_0 = \frac{1}{2} H_0$, and $h_1 = \frac{1}{2} H_1$. Hence

$$H_0^2 - H_1^2 = \frac{2}{g} (H_1 u_1^2 - H_0 u_0^2).$$

$\frac{1}{2} (H_0^2 - H_1^2) = \frac{1}{g} [H_1 u_1^2 - H_0 u_0^2]$. But, since $\Omega_0 u_0 = \Omega_1 u_1$, we have $u_1^2 = u_0^2 \frac{H_0^2}{H_1^2}$ and $H_0^2 - H_1^2 = \frac{2 u_0^2}{g} \left[\frac{H_0^2}{H_1} - H_0\right]$

* * * * * * *

"Dividing by $H_0 - H_1$, the equation becomes

$$\frac{H_1}{H_0} (H_0 + H_1) = \frac{2 u_0^2}{g};$$

therefore

$$H_1 = \sqrt{\frac{2 u_0^2 H_0}{g} + \frac{H_0^2}{4}} - \frac{H_0}{2}."$$

The last equation, as printed in the *Encyclopedia Britannica*, contains a typographical error, the sign of the term, $\frac{H_0}{2}$, being there printed +.

A comparison of the Merriman, Engels, and Unwin formulas as given herein, will show that they are essentially different, and will not give the same result for any given set of data.

The writer hesitates to make what might be considered a criticism of the work of the eminent engineers whose names accompany the foregoing formulas, and what follows has been written in an endeavor to find the reason for the differences between them.

Referring to the development of the Engels formula, which he refers to as an approximate one, it seems as though the difference between the velocity heads should be equal to the difference between the heights of the centers of gravity of the water masses, in the two phases of flow, rather than equal to the difference of elevation of the two water surfaces. Thus, we should have $\frac{v_1^2 - v_2^2}{2g} = \frac{d_2 - d_1}{2} = \frac{j}{2}$.

This expression is also obtained by equating the loss of kinetic energy in any water mass, M, which is $\frac{1}{2} M (v_1^2 - v_2^2) = \frac{W}{2g} (v_1^2 - v_2^2)$, to the work done in raising the same mass through the height, $\frac{1}{2} (d_2 - d_1)$, which is $\frac{W}{2} (d_2 - d_1)$. It is also obtained from Professor Merriman's formula by omitting the term which expresses the head lost in expansion.

If this expression is used, the formula for height of jump will be:

$$j = d_2 - d_1 = \frac{v_1^2}{2g} + \sqrt{\frac{v_1^2}{g} d_1 + \left(\frac{v_1^2}{2g}\right)^2} - d_1 \ldots\ldots (A)$$

Mr. Andrews.

It appears as though Professor Unwin, in the development of his formula, has treated static pressure as if it were a dynamic force. He takes the difference of static pressures on planes, *ab* and *cd*, multiplies it by the time, *t*, calls the product an impulse, and equates it to change of momentum.

The expression of this operation is $Ft = M(v_1 - v_0)$, in which F must be a dynamic force which causes the change of momentum from Mv_0 to Mv_1. To put the equation into what is perhaps a more common form, we may multiply both members by $\frac{v_1 + v_2}{0}$, which will change it to a work equation. Thus, $Ft\left(\frac{v_1 + v_0}{2}\right) = \frac{M}{2}(v_1^2 - v_0^2)$, and in this, $\frac{v_1 + v_0}{2}t$ represents the distance through which the force, F, acts on the mass, M, and $\frac{M}{2}(v_1^2 - v_0^2)$ represents the difference of kinetic energies possessed by the mass, M, after and before the action of the force, F. In the case of the flow of water in a conduit, it does not seem reasonable to think of a difference between two static pressures acting on the fluid through any particular distance, and thus doing work on the water, any more than differences of static pressures on moving water in a pipe might be considered to be doing work on the water as it flows.

Table 3 contains the data of Bidone's experiments, as given by Professor Merriman, together with values of j computed by the various formulas.

TABLE 3.—Data of Bidone's Experiments, as Given by Professor Merriman.

d_1.	v_1.	Observed j.	j computed by Merriman's formula.	j computed by Engel's formula.	j computed by Unwin's formula.	j computed by Equation (A).
0.149	4.59	0.274	0.290	0.288	0.224	0.632
0.154	4.47	0.267	0.283	0.270	0.213	0.595
0.208	5.59	0.405	0.428	0.435	0.329	0.940
0.246	6.28	0.493	0.531	0.556	0.417	1.190

It is quite evident that Equation (A), which assumes no loss of head because of eddies, does not give correct values of j, and that it should contain a term which will express such loss of head. The term inserted by Professor Merriman was of the form used in the case of sudden expansion of section of pipe: whether this or some other expression should be used may probably not be decided from theoretical considerations alone. It is conceivable that some forms of the wave may result in very little disturbance by eddies, just as some forms of

Mr. Andrews. pipe enlargement produce very little loss of head, and in these cases the jump should be higher than in those cases where many eddies are produced.

Mr. Johnson. R. D. JOHNSON,* ESQ.—The speaker has been much interested in Mr. Kennison's able and valuable paper, and is glad to take part in the discussion.

Mr. Kennison calls attention, in a novel way, to a well-known phenomenon of Nature. His treatment of the subject is conclusive, so far as it goes, and leaves little or nothing to be said in criticism. The speaker has always regarded these "alternative" or complementary levels in a different way from that presented by Mr. Kennison, and it may be worth while to view the matter in a somewhat more commonplace light than through the medium of a cubic equation, two of the roots of which not only demonstrate the existence of "alternative" stages, but also determine them in relation to the quantity of water flowing and the fall of potential from the static level.

A casual reading of the paper might lead one to conclude that the depth of water in a flume is an unstable affair, and that, with slight provocation, it may flop to its "alternative" and continue indefinitely to flow at a different depth. This is a correct theoretical conclusion, neglecting friction, but it might be interesting to show, in a rough way, how much it is affected ordinarily by a consideration of "slope". It may be interesting, also, to point out the simple relation of one "alternative" stage to the other, which does not involve the cubic equation, although no other method can express the whole story so completely in one equation.

For the sake of this argument, let us deal, in imagination, with a flume so wide that d and r may be regarded as interchangeable quantities, and, merely for the sake of simplicity in illustration, let C, in the old Chezy formula, be considered constant through a reasonable range of conditions. Let the flume have a smooth bell-mouth entrance in which friction may be neglected. Let the water be regarded as flowing at a uniform depth, d_1, and with a velocity, v_1, the slope of the flume being s_1. Now, at some point well along in the flume, let the slope suddenly change to a greater one, s_2. This condition would naturally cause an acceleration of the water, and if the relation of s_1 to s_2 were just right, and a suitable submerged weir were placed at the transition point, the depth, d_1, on the slope, s_1, would not be altered, but the water would "jump" or suddenly slide into another régime of flow on the slope, s_2, at another uniform, though shallower, depth, d_2, and at higher velocity, v_2. This relation of the two depths is the one so completely expressed by Mr. Kennison in his cubic equation. It may be shown that this exact condition, as described, can

* New York City.

Mr. Johnson.

occur only when the two slopes bear a definite relation to each other, more or less independent of the quantity, Q, and when they are also inversely proportional to the cube of the respective depths, or directly proportional to the cube of the respective velocities. Also, that s_1 is less than $\frac{g}{C^2}$, and s_2 is greater than $\frac{g}{C^2}$, and that when each is made equal to $\frac{g}{C^2}$, and the submerged weir is finally removed, the flume will be carrying all the water that can possibly enter it, and the depth, at this time being equal to $\frac{v^2}{g}$, has no "alternative" and, therefore, cannot be made to "jump" to another value.

The relation of the velocities and depths on either side of the transition point, previously referred to, is expressed simply as follows (neglecting local eddy losses):

$\frac{v_2^2 - v_1^2}{2g} = d_1 - d_2$; also, $v_1 d_1 = v_2 d_2 = Q$ per unit width.

From these equations, we have,

$$v_1^2 (d_2 + d_1) = 2 g d_2^2 \quad \ldots\ldots(1)$$

and,

$$v_2^2 (d_2 + d_1) = 2 g d_1^2 \quad \ldots\ldots(2)$$

or,

$$d_1 = \frac{v_2^2}{4g} \left\{ 1 + \sqrt{1 + \frac{8 g d_2}{v_2^2}} \right\} \quad \ldots\ldots(3)$$

and,

$$d_2 = \frac{v_1^2}{4g} \left\{ 1 + \sqrt{1 + \frac{8 g d_1}{v_1^2}} \right\} \quad \ldots\ldots(4)$$

These equations express the relation of d_1 to d_2 for a given quantity of water (d_2 v_2 or d_1 v_1) flowing, and may be written equally well in terms of Q, eliminating v_1 and v_2.

The Chezy formula gives:

$$v_1^2 = C^2 d_1 s_1, \text{ and } v_2^2 = C^2 d_2 s_2.$$

Dividing one by the other, we have,

$$\frac{s_1}{s_2} = \frac{v_1^2 d_2}{v_2^2 d_1} = \frac{v_1^3}{v_2^3} = \frac{d_2^3}{d_1^3},$$

which expresses the necessary definite relation of the two slopes, in order that the quantity of water flowing at uniform depths may be the same in both cases. The relation of one slope to the other, necessary to maintain, indefinitely, the "alternative" water levels, may be obtained by combining these equations with Equations (3) and (4), eliminating the v and d, and becomes $\left(\frac{g}{C^2} \text{ being put} = s_m\right)$:

$$s_1 = \frac{1}{s_2^2} \left\{ \frac{4 s_m}{1 + (1 + 8 s_m \div s_2)^{\frac{1}{2}}} \right\}^3 \text{ less than or equal to } s_m,$$

Mr. Johnson.

and, $s_2 = \frac{1}{s_1^2}\left\{\frac{4\,s_m}{1+(1+8\,s_m \div s_1)^{\frac{1}{2}}}\right\}^3$ greater than or equal to s_m.

These particular slopes might appropriately be termed "alternative" slopes.

By inspection of Equation (1), it may be seen what takes place when s_1 is increased and s_2 is decreased, bringing them together and drowning out the "jump"; for then d_1 and d_2 become equal, and we have, calling the new common velocity, v_m, and the depth, d_m,

$$v_m^2 \times (2\,d_m) = 2\,g\,d_m^2, \text{ or } d_m = \frac{v_m^2}{g}, \text{ and } s_m = \frac{g}{C^2},$$

which might be termed the "natural slope" of a flume, because it represents the minimum slope which will carry away, at uniform depth, all the water which can be accelerated into it or be made to enter it. The depth of the water is twice the velocity head, and, therefore, the surface at the flume entrance drops one-half the depth in order to accelerate the water. As Mr. Kennison has pointed out, this condition obtains when the flume offers no restriction to the flow, and the maximum possible quantity is entering it; or, in this case, the point of control caused by the submerged weir has been entirely removed, and the slope evened up uniformly throughout its length.

If the distance from the still-water surface to the bottom of the flume, at the entrance, be called H, then $d_m = \frac{2}{3}H$, and $v_m = \sqrt{2\,g\,\frac{H}{3}}$, and $Q = d_m\,v_m = \frac{2\,H}{3}\sqrt{\frac{2}{3}\,g\,H} = 3.09\,H^{\frac{3}{2}}$, which is the natural formula for a smooth submerged weir, and also represents the quantity which requires a slope of $\frac{g}{C^2}$ to keep it moving at uniform depth, and expresses a stable condition of flow which has no "alternative".

The foregoing analysis falls far short of a rigid and logical mathematical discussion, and is intended only to point out some interesting physical relations which may furnish a side light on Mr. Kennison's work, even if it adds nothing to it.

The fact that C is not constant and that r is not equal to d will modify, of course, the numerical results derived for any particular case.

Lately,* there has been so much discussion of the mysteries of the hydraulic jump that the writer feels impelled to advance his own theory of the nature and magnitude of the losses involved, because he believes that the matter is capable of a more or less accurate analysis and should therefore be particularly interesting to those students of the subject who have searched in vain for an insight

* This part of Mr. Johnson's discussion was submitted subsequently by letter.

into Nature's method of determining a new water level following the so-called jump. Mr. Johnson.

In the discussions on this paper there appear many conflicting arguments, not only as to the possible magnitude of the losses, but even as to whether or not any loss exists—and it seems a proper time to lay down an exact method of determining these losses and to prove it by harmonizing the momentum and energy equations involved.

In the first place, it may be positively stated that, in a channel of uniform width, water cannot pass without loss from a lower to a higher level if the bottom remains flat. In other words, it may easily be proved essential that the lifting of the water surface be accompanied by a humping up of the bottom, if major eddy losses are to be eliminated. This will clearly appear from a brief mathematical discussion, but first it is desired to point out a fundamental law which pertains to the flow of water in a way perfectly analogous to its behavior with respect to solid bodies.

It is a well-known principle of impact that the sum of the momenta of two solid bodies cannot be altered by such bodies merely bumping into each other, and this is true, whether or not the impact results in a loss of energy; this law is commonly called the conservation of momentum, and is particularly interesting as applied to fluids because this persistence of the momentum law, in spite of energy or eddy losses, enables a comparison to be made between the momentum and energy equations which may disclose the inevitable eddy loss and determine the exact amount of it.

Now, in the case just referred to, of a change of water level to its higher alternative, theoretically without loss of energy, a comparison between the momentum and energy equations clearly shows the necessary existence of a force, F, which represents the tendency of the weir to slide up stream, and the necessary existence of the submerged weir is demonstrated in this, if in no other way, because the force cannot exist without the presence of the obstructing body.

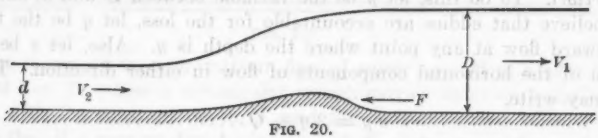

FIG. 20.

There are many other interesting ways of demonstrating the same thing, but they will be passed over as not particularly pertinent to the matter in hand.

Referring to Fig. 20, the energy equation is as follows, if there is to be no eddy loss:

$$\frac{v_2^2 - v_1^2}{2g} = D - d \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

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The momentum equation, which always holds good, whether or not there are energy losses, for unit weight of fluid and unit width of flume, is as follows:

$$\frac{D^2 - d^2}{2} - F = \frac{Q}{g}(v_2 - v_1) \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

which has been referred to by Dr. Unwin as a statement of the well-known physical law that force is equal to the rate of change of momentum.

It will be seen, from these two equations, remembering the relation, $d\,v_2 = D\,v_1 = Q$, that the value of F may be expressed in terms of other known quantities, and can never be zero, if Equation (1) holds good; therefore, Equation (1) cannot express the relation between the two depths and velocities unless a weir of proper form and dimensions is interposed.

Having now demonstrated the existence of the force, F, let us see what happens in the case of the hydraulic jump when there is no weir, and when, therefore, F must be zero.

It may be observed that by re-writing Equation (1) so as to include a certain friction or eddy loss, f, then the value of F in Equation (2) may become zero—and the new equations are:

$$\frac{v_2^2 - v_1^2}{2g} = D - d + f \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

and,

$$\frac{D^2 - d^2}{2} = \frac{Q}{g}(v_2 - v_1) \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

from which f may be determined as in the former case.

Now, although this statement of the matter really completes the whole story, it may be interesting to integrate the total eddy loss in a manner completely independent of Equation (3), and, having obtained its value, to show that its inclusion in Equation (3) makes this equation identical with Equation (4), thus demonstrating the whole matter *a fortiori*. To do this, let y be the variable between D and d, and, as we believe that eddies are accountable for the loss, let q be the total backward flow at any point where the depth is y. Also, let v be the mean of the horizontal components of flow in either direction. Then we may write,

$$v\,y = 2\,q + Q \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

and, from the momentum principle,

$$\frac{y^2 - d^2}{2} = \frac{Q}{g}(v_2 - v) \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Solving $2\,q$ in terms of y, we have,

$$2\,q = \left(v_2 + \frac{g\,d^2}{2\,Q}\right) y - \frac{g\,y^3}{2\,Q} - Q \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

Mr. Johnson.

Now, the power required to force the water, q, against the differential pressure, dy, is qdy; but, as the same quantity of water is returning on the other side of the eddy due to the same difference of head, dy, and under identical uniform conditions of impact, or heat losses, the total power required to keep these eddies in motion must be 2 qdy, and therefore the total lost power is expressed as

$$2\int_d^D qdy, \text{ and the friction head, } f = \frac{2}{Q}\int_d^D qdy.$$

Expressing q in terms of y, as above, and integrating we find,

$$f = \frac{D^2 - d^2}{2\,Q}\left(v_2 - \frac{g}{4\,Q}(D^2 - d^2)\right) - (D - d) \ldots\ldots\ldots (8)$$

and, if this is substituted in Equation (3) and the equation simplified as far as possible, we shall have Equation (4) as a result, thus harmonizing the equations of energy and momentum by recourse to a rational hypothesis of eddy loss.

Mr. Merriman.

MANSFIELD MERRIMAN,* M. AM. SOC. C. E.—Fig. 11 of Mr. Kennison's interesting paper appears to indicate that the hydraulic jump is caused by a slight obstruction or irregularity in the bed of the channel just below the toe of the jump; this figure also shows an upward spurt of the water at the top of the jump. The speaker, having seen jumps formed in a rectangular trough, is able to affirm that they may occur without any obstruction near the toe, and that usually there is no upward spurt at the top. Undoubtedly, a so-called jump may occur when a considerable obstruction exists in the bed of the channel and perhaps such was in the mind of the author when he prepared Fig. 11. The true hydraulic jump, however, is of a different character.

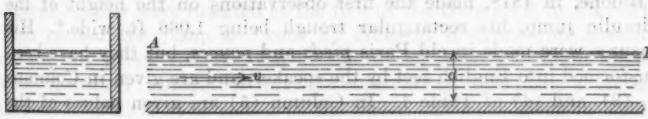


FIG. 21.

The theory of the non-uniform flow of water applied to a case like that of Fig. 21, where a rectangular trough carries water on a uniform slope, with the constant depth, d, and the constant mean velocity, v, shows that if a gate or dam be inserted at E, a change in the surface, $A\,E$, will take place in one of two ways. Fig. 22 shows the way which occurs when v is less than $\sqrt{g\,d}$, where g is the acceleration of gravity. Here the water surface is convex to the bed of the channel and extends a long distance up stream. This is the common case of backwater

* New York City.

Mr. Merriman.

which occurs in streams when the original surface of the water is raised by the construction of a dam or bridge piers.

Fig. 23 shows the second case, where the mean velocity, v, is greater than $\sqrt{g\,d}$, and here the true hydraulic jump is formed. From A to B the water is flowing with the uniform depth, d, at B the depth suddenly increases and shortly after reaches the greater depth, D, the surface of the water from B to E being concave to the bed of the channel. The distance, $E\,B$, from the dam back to the jump depends on the velocity, v, only; when v is only slightly greater than $\sqrt{g\,d}$, the distance, $E\,B$, is large; when v is very much greater than $\sqrt{g\,d}$, the distance, $E\,B$, may be quite small; but, wherever the jump may be formed, the depth, D, is theoretically always the same for the same velocity, v, and the same depth, d.

FIG. 22.

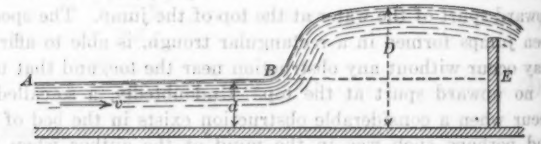


FIG. 23.

Bidone, in 1818, made the first observations on the height of the hydraulic jump, his rectangular trough being 1.066 ft. wide.* His measures were made in old Paris *pieds* and *pouces*, but they have been transformed into English feet by the speaker, and are given in Columns (2), (3), and (4) of Table 4. In Column (5) are given values of the ratio, $\frac{v}{\sqrt{g\,d}}$, as computed by the speaker, and in Column (6) are values of the lost head, computed from the expression,

$$h' = \frac{v^2 - V^2}{2\,g} + d - D,$$

in which V, the velocity corresponding to D, is found from $V = \frac{v\,d}{D}$, while g has been taken as 32.19 ft. per sec. per sec., this being the value for Paris, where the observations of Bidone were made.

* *Transactions*, Royal Soc. of Turin, 1819, pp. 21–80.

TABLE 4.—BIDONE'S OBSERVATIONS ON HYDRAULIC JUMPS.

(1) No. of experiment.	(2) Observed mean velocity, v, in feet per second.	(3) Observed mean depth, d, in feet.	(4) Observed mean depth, D, in feet.	(5) Computed ratio, $\frac{v}{\sqrt{g\,d}}$.	(6) Computed loss of head, h', in feet.
I_1	4.47	0.155	0.423	2.0	0.001
I_2	4.45	0.155	0.437	2.0	+ 0.013
I_3	4.41	0.156	0.430	2.0	− 0.012
I_4	4.52	0.152	0.436	2.0	− 0.005
II_1	5.59	0.208	0.613	2.2	0.024
II_2	5.56	0.210	0.620	2.1	0.015
II_3	5.52	0.211	0.630	2.1	0.002
II_4	5.49	0.212	0.642	2.1	− 0.013
II_5	5.67	0.206	0.647	2.2	0.007
III_1	6.29	0.246	0.738	2.2	0.054
III_2	6.36	0.244	0.745	2.3	0.059
III_3	6.39	0.242	0.764	2.3	0.049
IV_1	4.55	0.150	0.398	2.1	0.028
IV_2	4.57	0.150	0.405	2.1	0.040
IV_3	4.57	0.150	0.428	2.1	0.007
IV_4	4.59	0.149	0.423	2.1	0.013

In 1894, a number of observations were made at Lehigh University, by Mr. Robert Ferriday, in the preparation of a graduating thesis. An abstract of this thesis was published,* but, as it gives no results of the observations, it seems best to put them on record here, since the work was done under the writer's supervision. Mr. Ferriday's trough was 0.66 ft. wide and was laid on various slopes so as to produce different velocities. The depth in each observation was determined from the mean of three measures, and the velocities were found by measuring the water caught in a barrel in a given time. Dams of different heights were inserted at the foot of the trough (E in Fig. 23), which caused the jump to occur at different distances up stream; but, for any given velocity, the height of the jump was found to be closely the same. Altogether forty-six observations were made at eleven different velocities. The results of the eleven series are given in Table 5, in Columns (3), (4), and (5); and Columns (6) and (7) give the computed values of the ratio, $\frac{v}{\sqrt{g\,d}}$, and the head, h', lost in making the jump. In these computations, the value of g for Lehigh University has been taken as 32.16 ft. per sec. per sec.

These observations on the height of the hydraulic jump are all that are known to the speaker, except those made by Darcy and Bazin in 1856.† They give several series of observations made in a rectangular timber conduit, 1.99 m. (6.53 ft.) wide. In the series

* *Engineering News*, July 11th, 1895, Vol. 34, p. 28.

† "Recherches Hydrauliques" (Paris, 1865), pp. 284–292.

Mr. Merriman.

numbered 89, 90, and 91, the jump was quite ill-defined on account of the slight excess of v over $\sqrt{g\,d}$, the ratio of the former to the latter being from 1.1 to 1.3. For Series 92, the ratio, $\frac{v}{\sqrt{g\,d}}$, was between 1.5 and 1.9, and several characteristic jumps were obtained. All observations made by Bidone, however, have values of this ratio ranging from 2.0 to 2.3, and in six of Ferriday's observations the ratios are from 3.1 to 3.9. It appears unnecessary to give the observations of Darcy and Bazin here, as they may be readily consulted in their book. The speaker is inclined to the opinion that good characteristic jumps will rarely be formed unless the velocity, v, is greater than $2.0\ \sqrt{g\,d}$. The great variation in the values of the lost head given in the last columns of Tables 4 and 5, shows how difficult it is to make precise measurements on the height of the hydraulic jump; five of these values are negative, which indicates that the corresponding observations are probably of little value.

TABLE 5.—FERRIDAY'S OBSERVATIONS ON HYDRAULIC JUMPS.

(1) No. of series.	(2) No. of observations in series.	(3) Observed mean velocity, v, in feet per second.	(4) Observed depth, d, in feet.	(5) Observed depth, D, in feet.	(6) Computed ratio, $\frac{v}{\sqrt{g\,d}}$.	(7) Computed loss of head, h_r, in feet.
I	3	2.18	0.050	0.143	1.7	0.028
II	5	2.98	0.044	0.150	2.5	+0.020
III	6	3.56	0.036	0.153	3.3	0.009
IV	5	3.66	0.063	0.168	3.5	0.005
V	3	4.39	0.095	0.267	2.5	0.090
VI	4	5.02	0.068	0.285	3.1	0.155
VII	3	5.02	0.071	0.290	3.3	0.151
IX	4	3.50	0.055	0.162	2.0	0.061
X	4	3.95	0.046	0.175	3.2	0.097
XI	5	4.06	0.042	0.188	3.5	0.097
XII	4	4.33	0.038	0.205	3.9	0.114

Mr. Ferriday also noticed the fact that at the toe of the jump there was no horizontal velocity along the bottom of the trough, as balls of putty placed in the trough at the upper end moved along the bottom until they reached a point just below the toe of the jump, where they stopped. This indicates that probably the lower strata of water turn upward in order to make the jump; if so, it may be inferred that a jump formed near the foot of an apron of an ogee dam will be very advantageous in preventing scour.

Two formulas for the height of the jump are given by Bidone in his paper of 1819, and four or five others have been derived by later writers. All these formulas are imperfect on account of the neglect of the fall between the sections at d and D and other ele-

ments difficult to include, and also, in some cases, on account of defective reasoning. The formula of Belanger, later adopted by Rühlmann and by Weisbach, assumes that no loss of head occurs in forming the jump, and hence values computed from it are considerably in excess of observed values. Nearly all formulas, in fact, give computed values higher than those observed. To derive a reliable formula for the height, D, from observed values of d and v, it seems necessary that coefficients should be introduced, which are derived from the study of observations. No doubt this will be done in time, and, perhaps, the observations previously given may aid in deriving such coefficients. Mr. Merriman.

The author's use of the term "hydraulic gradient" seems to be unfortunate, and does not agree with that generally used. When a number of piezometers are placed on a pipe, the line connecting the water levels in them is the hydraulic gradient; when water flows in an open channel, its surface is the hydraulic gradient. The term "energy gradient" better applies to the curve formed by successive values of the author's H, as this is the sum of the pressure head and the velocity head.

The author deserves much credit for preparing this interesting paper, as it deals with a subject which has been rarely discussed. Observations like those made by Mr. Ferriday, in 1894, can easily be conducted in any hydraulic laboratory, and it is hoped that further experimental work may be done in this direction. One who has seen the true hydraulic jump, under the conditions shown in Fig. 23, can never forget it, as it violates the impressions which he has formed by observing the usual backwater case of Fig. 22. In all hydraulic laboratories for the instruction of students it would be well to have the hydraulic jump exhibited occasionally.

E. G. WALKER,* Assoc. M. Am. Soc. C. E. (by letter).—The problem of the "hydraulic jump", or, as it is frequently named, the "standing wave", is one which has been before irrigation engineers for many years. The phenomena accompanying the formation of a standing wave were investigated by the engineers of the Government of India in the early days of river control and irrigation works in that country. As the author states, one may look in vain in the generality of textbooks on hydraulics for a proper discussion of the subject, but the writer wishes to point out one important exception which, apparently, has not been brought to Mr. Kennison's notice. In his article on "Hydraulics" in the Ninth Edition of the Encyclopædia Britannica, Dr. W. C. Unwin, at that time Professor of Hydraulic and Mechanical Engineering at the Royal Indian Engineering College, Cooper's Hill, developed the theory of the standing wave on a rational basis, and Mr. Walker.

* Hull, England.

Mr. Walker.

worked out the conditions under which the phenomenon becomes possible.* This is the most complete exposition of the subject known to the writer. The article has been revised for the present (Eleventh) edition of the Encyclopædia.

The author, in Section 2 of the paper, starts off his theory with a postulate that "there are certain controlling sections" at which "for the given head and channel depth, the discharge is a maximum." Some further explanation of this statement appears to be desirable, for as soon as a definite régime of flow exists in a channel, no matter how complex the conditions of flow may be, the discharge at all sections must be the same, or there would be discontinuity of flow at some point. Would it not be a truer statement of the case to say that there exists a "critical stage" in the flow, dependent on the friction, form and slope of the channel bed, and the velocity of the stream, at which the water level will change from the lower to the higher?

The statement in the last paragraph of Section 2 is important. Considerable misconception has arisen owing to the incorrect application of formulas for the loss of head by sudden expansion to conditions to which they are not applicable. The sudden transition from one state of equilibrium to the other, found in the hydraulic jump, involves no energy loss, except such as is due to the fact that water is not exactly the theoretical fluid it is assumed to be. There is no reason for assuming, without proof, that results applicable to flow in pipes should be applied without change to open-channel flow under atmospheric pressure. It has been shown experimentally that such assumption is incorrect.

Although the author discredits the "sudden expansion" theory as originally propounded, he does not offer any modification to replace it. On the principle so often used in hydraulic calculations, namely, that force exerted is equal to the rate of change of momentum, Dr. Unwin develops, in the article previously mentioned, a formula for the depth of water after the jump, which is free from incorrect assumptions. His formula, in the notation of the paper, is

$$d_2 = \sqrt{\left\{ \frac{2\,V_1^2}{g}\,d_1 + \frac{1}{4}\,d_1^2 \right\}} - \frac{1}{2}\,d_1 \qquad \ldots\ldots(1)$$

as compared with,

$$d_2 = \sqrt{\frac{2\,V_1^2}{g}\,d_1} \qquad \ldots\ldots(2)$$

the formula of the "sudden expansion" theory.

The subject has recently been investigated experimentally by Professor A. H. Gibson, of Dundee, Scotland. His experiments are described in a paper entitled "The Formation of Standing Waves in

* Vol. XII, pp. 499–502.

an Open Stream."* He applies Equations (1) and (2) to his experimental results, and also to a series of experiments of Messrs. Darcy and Bazin, and shows that, for his own experiments, whereas Equation (2) gives discrepancies of from 3 to 23% excess in the values of d_2, the range of variation using Equation (1) is only from $-4\frac{1}{2}$ to $+3\frac{1}{2}$ per cent. The results in the case of the Darcy and Bazin experiments are more startling, the differences being, for Equation (2), from 22 to 77%, and for Equation (1) from — 3 to + 3 per cent. By allowing for the variation of velocity over the cross-section of the stream, the depth calculated from Equation (1) can be brought still more closely to approximate to the measured depths. Mr. Walker.

EDWARD W. BUSH,† M. AM. SOC. C. E. (by letter).—Figs. 24 to 28 may be of interest to those studying the hydraulic jump below ogee-faced dams, where the toe is some distance up from the bed of the Mr. Bush.

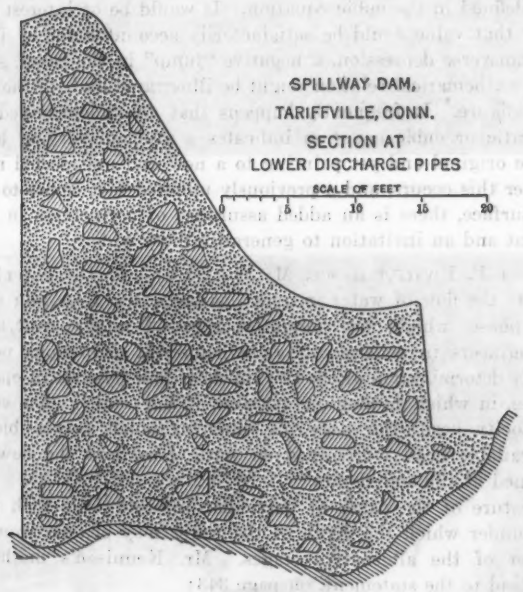


FIG. 24.

stream. This dam is on the Farmington River, at Tariffville, Conn., and was completed late in the season of 1899, by the Hartford Electric Light Company. On February 14th, 1900, when the flood pictures

* *Minutes of Proceedings*, Inst. C. E., Vol. CXCVII, p. 233, *et seq.*

† Hartford, Conn.

Mr. Bush. (Figs. 26, 27, and 28) were taken, the flood height was 9.5 ft. above the crest, and 1.5 ft. above the bulkhead.

Fig. 24 shows the section of the dam at the lower discharge pipes; Fig. 25 is a construction view on September 24th, 1899; and Figs. 26, 27, and 28, show the 9.5-ft. flood passing over the dam.

The jump was quite irregular in action, and surged up and down stream over a considerable distance. The height of the jump also varied greatly.

Fig. 25 shows that the river bottom was quite irregular below the dam. This, no doubt, helped to cause the irregularities in the jump. No damage was caused by the flood. The bulkhead wall was raised a few feet to prevent future floods from over-topping it.

Mr. Dunham. H. F. DUNHAM,* M. AM. SOC. C. E.—In the early part of this discussion there was a reference to a value that was mysterious—a third root undefined in the cubic equation. It would be of interest to know whether that value could be satisfactorily accounted for by introducing a transverse depression, a negative "jump" in the canal, so that a further mathematical relation might be illustrated by both the analysis and the figure. It frequently happens that an undetermined root in a quadratic or cubic equation indicates a problem varying but little from the original and yet pointing to a new and unexpected meaning. Whenever this occurs, and a previously unnoted fact comes to light or to the surface, there is an added assurance of correctness in the first statement and an invitation to generalize more widely.

Mr. Ewald. ROBERT F. EWALD,† ASSOC. M. AM. SOC. C. E. (by letter).—With respect to the flow of water in open channels, Mr. Kennison discusses certain phases which have heretofore received rather scant attention from engineers in general. His mathematical deductions permit of the ready determination of the two limits of flow possible in open channels; one, in which as much as possible of the total head is converted into velocity head; the other, in which as much as possible of the total available head is retained as static head and the flow is still maintained at the stated rate.

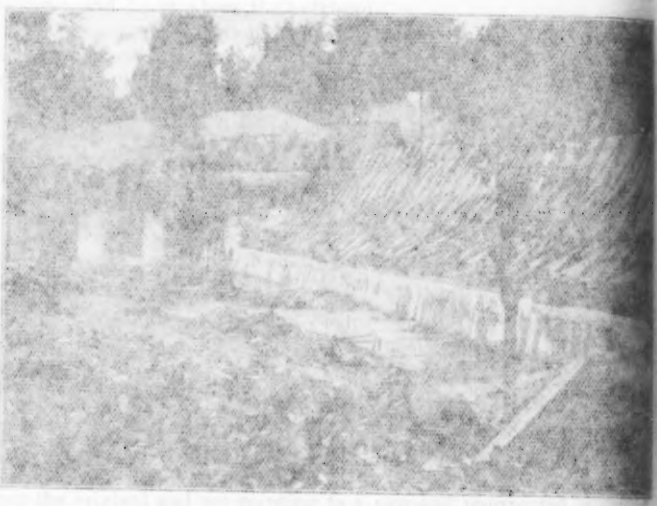
A feature of the paper of particular interest deals with the conditions under which the depth of the water may change from one to the other of the alternative stages. Mr. Kennison's mathematical studies lead to the statement, on page 343:

"The jump proper, or the passing from the lower to the upper stage, does not involve energy losses, except incidentally, and it is doubtful if the ordinary formula for loss by 'sudden expansion' applies in this case."

* New York City.
† Alcoa, Tenn.

FIG. 25.—DAM AT TARIFFVILLE, CONN., UNDER CONSTRUCTION SEPTEMBER 4TH, 1899.

FIG. 26.—TARIFFVILLE DAM, WITH 9.5-FOOT FLOOD.



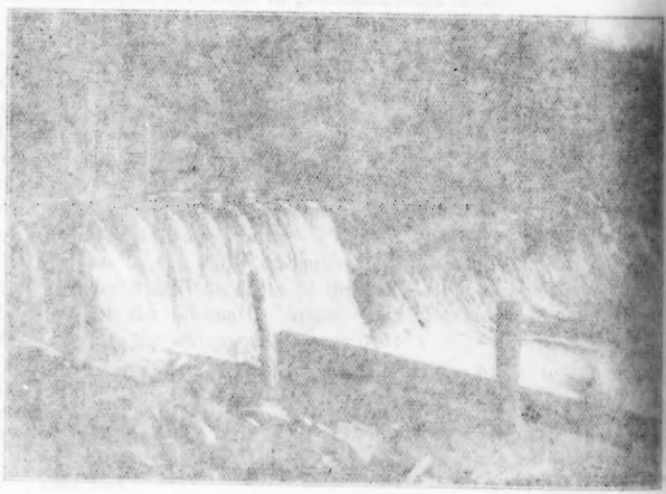
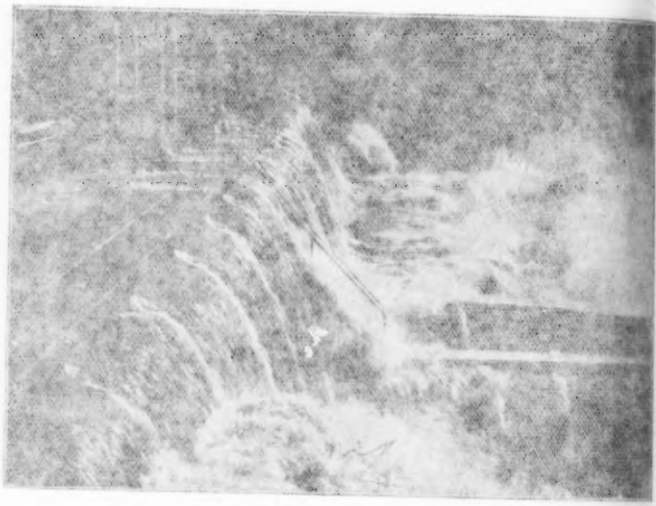


FIG. 27.—TARIFFVILLE DAM, WITH 9.5-FOOT FLOOD.

FIG. 28.—TARIFFVILLE DAM, WITH 9.5-FOOT FLOOD.



Mr. Ewald.

This statement may need modification, and perhaps should be strictly limited to rises occurring under conditions governed by actual external controlling sections, as defined in the paper, because there is ample evidence to show that, in channels of uniform cross-section and gradient, rises may be produced which are accompanied by considerable inherent energy losses, which losses are independent of channel conditions, and apparently conform to some law not yet definitely established. These energy losses, when very high velocities are concerned, amount to a very large percentage of the original kinetic head, and become a factor of the greatest importance. If the existence of inherent energy losses is admitted, then the jump, in a sense, becomes a control, because, within it, the hydraulic gradient is brought down to such a position as to permit of the change in levels. In order

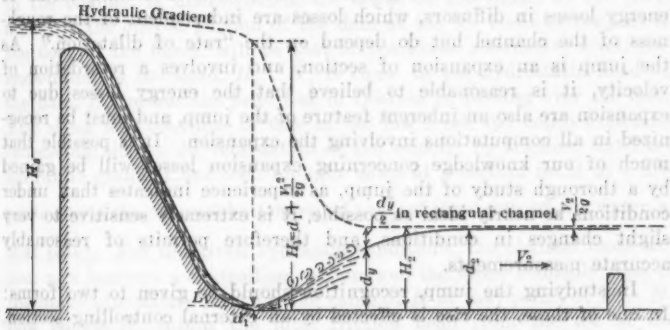


FIG. 28.

to render the actual conditions clearer, it will be necessary to refer to Fig. 29, which shows the ideal jump as developed at the toe of an overflow dam. An essential condition in this case is that the obstruction, or weir regulating the height of the jump, be far enough down stream to permit the water, after passing through the jump, to attain uniform flow parallel to the bottom before passing over the weir. Due to the inherent energy losses occurring in the water as it rises, the hydraulic gradient has been brought down from its high level at section d_1 to the much lower level at d_y. At the point, d_y, where the gradient becomes nearly level and the depth of the water in a rectangular channel is two-thirds of the distance from the bottom of the channel to the hydraulic gradient, we have a condition in which the water is flowing, according to Mr. Kennison's theory, at the lower of two corresponding alternative stages for the total head, H_2. The higher stage for this value of H_2 is that existing at the section, d_2,

Mr. Ewald.

and the water rises to this level from d_y without further loss of energy. Actually and theoretically, we have a condition then in which, discussing the jump as a whole, there is an inherent loss of energy, causing the drop of the hydraulic gradient and the establishment of a controlling section within the jump. The remainder of the rise is effected without considerable energy loss.

The writer includes the term "theoretically" in the previous paragraph, because he hopes that, ultimately, the science of hydromechanics will be advanced so that it will be possible to determine and explain, with a reasonable degree of accuracy, the losses of energy due to expansion, much in the same way as the electrical engineer is able to measure and foretell the effects of self-induction in alternating currents. Mr. F. zur Nedden, in his paper on "Induced Currents of Fluids,"* recognizes the existence of certain forms of energy losses in diffusors, which losses are independent of the roughness of the channel but do depend on the "rate of dilatation." As the jump is an expansion of section, and involves a retardation of velocity, it is reasonable to believe that the energy losses due to expansion are also an inherent feature of the jump, and must be recognized in all computations involving the expansion. It is possible that much of our knowledge concerning expansion losses will be gained by a thorough study of the jump, as experience indicates that, under conditions as nearly ideal as possible, it is extremely sensitive to very slight changes in conditions, and therefore permits of reasonably accurate measurements.

In studying the jump, recognition should be given to two forms: In one of these, the rise is effected by an external controlling section, and no energy losses occur, aside from those due to channel friction and a certain minor form of impact loss corresponding to that occurring in solid bodies and expressed by the term, $(1 - e)^2 \frac{(V_1 - V_2)^2}{2} M$, in which e is the coefficient of restitution. It is suggested that this form of the jump be designated as the "standing wave", as the term "wave" does not necessarily involve the idea of energy loss. (It will be noted that Mr. Groat has used the term "standing wave" in his discussion.) The second type of rise occurring in channels of uniform cross-section, involving certain inherent energy losses, and forming within itself the controlling section, may be designated as the "hydraulic jump", as the term has been used so frequently in connection with the discussion of energy losses at the toes of overfall dams, which cases usually involve rises of the type under discussion.

The Standing Wave.—The standing wave is formed when channel and external controlling sections are adjusted so as to produce flow

* *Proceedings*, Am. Soc. C. E., for August, 1915.

in accordance with the conditions discussed by Mr. Kennison and Mr. Groat, and constitutes a type of rise which has been observed frequently. Examples of the standing wave are given by Mr. Kennison in Figs. 5, 7, and 8.* The flow over submerged weirs under certain conditions affords the most and best examples. The standing wave has been so ably handled by the author and those following him, that no further discussion will be attempted. Mr. Ewald.

The Hydraulic Jump.—At the risk of being charged with presenting matters much too elementary, a brief discussion will be given of the action of the hydraulic jump under conditions as nearly uniform and ideal as it is possible to obtain. An ogee type, straight-crested, spillway dam is constructed in an open rectangular flume of dressed lumber. The cross-section of the flume is uniform, and the bottom is level. If water is allowed to pass over the dam, and the flow in the channel is unobstructed, the depth of the water, at the point at which the curved portion of the dam at the toe becomes tangent to the floor of the flume, will be that of the lower alternative level, corresponding to the head, H_1. H_1 is equal to the hydrostatic head on the dam, H_s, minus the head lost in friction in passing over the dam. The water will continue to flow at the low level, the depth gradually increasing, due to the retardation of the velocity by friction, until the hydraulic gradient has been brought down at a certain point to a distance above the channel bottom equal to $\frac{3}{2}$ of the depth of the water at that point. For the given hydraulic conditions, the channel cross-section now becomes a controlling section, and the water suddenly rises to the upper alternative level for the value of H at that point, or H_2. H_2 now equals H_1 minus the total friction head from the toe of the dam to this point. Where the velocity is high, a considerable length of the flume is required to lower the hydraulic gradient to the requisite height.

If, at a short distance toward the dam from the point where the rise is developed, a slight obstruction is placed across the channel, it is probable that sufficient energy will be absorbed in impact to bring the gradient down to the requisite height, and the wave will move up stream to or near the obstruction. Now, if slight increments are made to the depth of the water, the crest of the rise will move toward the dam, and finally the initial point may be brought exactly to the toe of the dam. The conditions are now as shown in Fig. 29. The obstruction in itself does not develop the energy losses, and, as the velocity of the water over the channel bottom has been decreased, the effect of friction is decreased materially. The losses necessary to bring the gradient down to the position shown are developed within

* One is also shown in the central portion of Fig. 358 in the first edition of Mead's "Water Power Engineering."

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the rise. Experiments of this character have been performed frequently, and there seems to be a definite relationship between the velocities, V_1 and V_2, and the depths, d_1 and d_2; hence the conclusion that the energy loss is inherent, and is probably a determinable quantity.

A few figures will be presented in order to indicate the extent of observed losses in the jump. On a dam, constructed in general according to Fig. 29, 4.08 ft. high above the floor of the flume, with a discharge of 0.580 cu. ft. per sec. per lin. ft. of crest, the depth of the water at d_2 was found to be 0.583 ft., and, with a Pitot tube, the velocity head at d_1 was found to be 3.17 ft., corresponding to a velocity of 14.29 ft. per sec. From the discharge, estimated from the head on the dam, which had been calibrated previously by a sharp-crested, suppressed weir, the depth of water at d_1 was estimated to be 0.041 ft. The value of H_1, then, must have been $3.17 + 0.041 = 3.211$ ft. For this value of H_1, according to Mr. Kennison, the lower level depth should be 0.039, which checks out reasonably well with the observed figure, and the upper level depth would have been 3.2103. The obstruction, a movable weir crest, was placed 18 ft. down stream from the toe of the dam. Without any change in the character of the channel, at the point where the expansion occurred, an actual loss of head, amounting to $(3.17 - 0.155) - (0.583 - 0.041) = 2.473$ ft., took place within a distance in which, previously, not more than 0.01 had occurred, with a velocity of 14.29 ft. per sec. The loss amounted to 82.3% of the difference in velocity heads between d_1 and d_2, and 79.6% of the original velocity head.

Many of those who have read Mr. Kennison's paper, no doubt have looked up the description of the Truckee main canal chute, and read the account of the jump formed therein. Judging from the illustrations accompanying the description, at least 15 ft., or about 85% of the original velocity head, was absorbed in the jump. As this occurred in a relatively smooth channel, it is extremely difficult to conceive of any form of channel friction or other form of "incidental loss" that would account for the great loss of head that actually developed.

The various pictures of dams in action, in the paper and in the discussions, give good visual evidence of the existence of large energy losses. In the case of the Bassano Dam, the fact has been pointed out by Mr. Muckleston that channel conditions, as influenced by the presence or absence of irregularities designed to increase impact losses, have relatively insignificant influence on the amount of head absorbed.

Where the jump is formed in a regular channel, on smooth surfaces, similar to the conditions for the dam in the flume, as just

Mr. Ewald.

discussed, casual observation and simple tests reveal several interesting features. The quantity of water undergoing actual retrograde movement toward the dam is relatively small as compared with the total quantity flowing. A small quantity flows down the surface of the rise sloping toward the dam, and produces the appearance of considerable agitation. Pitot tube tests show that the velocities are almost always down stream, and decrease fairly uniformly. The absorption of energy, under these conditions, is accompanied by a minimum of sound, the sensation on the observer being that gained by listening to a good motor running at full load under the most efficient conditions, after the starting noise has subsided. The general impression is that fairly complete absorption of energy is taking place, without much influence on the channel. These remarks, of course, do not apply to channels in which irregularities of any considerable extent are to be found.

There is serious need of reasonably exact knowledge concerning the nature and extent of the energy losses involved in the jump. In connection with the design of dams, the jump, in itself, establishes the minimum hydraulic conditions under which the requisite expansion of section and reduction of velocities to safe limits may be accomplished. The position of the jump must be controlled at all stages, so that the expansion is effected on smooth aprons of material which will resist the initial velocity. This is true, not only of low dams and waste-ways, but also of high dams, in many of which the overfall velocities pass the limits of those which may be safely carried by the hardest rock. The necessity and dimensions of such aprons are matters which should be capable of ready determination while the dam is being designed. At present, there are no accepted rules or hydraulic formulas which will enable one to assert positively, in all cases, that the existing normal channel depth of water is or is not sufficient for the formation of the jump at the toe of the dam. Fortunately for most dams, the natural channel depths down stream from the dam are sufficient to maintain depths greater than necessary, and also, in most cases, the rise of the water below the dam is at a greater rate than on the crest, usually producing more stable conditions at the extremely high discharges than may be found at the intermediate stages. A particular case of this kind is pointed out by Mr. Muckleston.

It has been assumed, generally, that the mathematical expression for the lost head in the jump has the general form, $\frac{(V_1 - V_2)^2}{2g}$. This form probably is not far from the truth, when it is remembered that the work involved in retarding the velocity of a moving body

Mr. Ewald.

from V_1 to V_2 ft. per sec., according to the principles of mechanics, has the general form, $\frac{M}{2}(V_1 - V_2)^2$. As the losses are partly effective in retarding the velocity of the water passing through the jump, it seems probable that the general expression for the loss will be a term of the form, $\frac{(V_1 - V_a)^2}{2g}$, in which V_a is to be considered as the average velocity that the water would have, assuming that the losses alone are effective in retarding the velocity. The final value of the velocity, V_2, has been used by several writers for V_a, but this, apparently, is inconsistent with the original assumption, as part of the reduction to the final velocity, V_2, is effected, during the vertical component of the path traversed by each particle of water, in direct action against the force of gravity, and the energy represented by this process reappears as static head. This is not lost head. The term, $\frac{(V_1 - V_a)^2}{2g}$, may also be expressed in the form, $\frac{\left[V_1 - (\sqrt{2gh'} + V_2)\right]^2}{2g}$, in which $\sqrt{2gh'}$ is to be considered as the average velocity retarded in rising against the "force of gravity" alone through the height, $d_2 - d_1$. The correct evaluation of the term, $\sqrt{2gh'}$, is necessary in order to obtain the true general equation for the jump. The true relation of the values of V_a or $\sqrt{2gh'}$ to the other factors entering into the jump, such as V_1, V_2, d_1, or d_2, has not yet been established. The limiting value of $\frac{(V_1 - V_a)^2}{2g}$ apparently is an expression of the form $\frac{(V_1 - k\,V_2)^2}{2g}$, in which $k\,V_2$ is to be considered as the velocity at that point in the jump at which the depth of water bears such a relationship to the height of the hydraulic gradient that the further rise of the water to the higher level down stream from this point is effected without further loss of energy, in accordance with Mr. Kennison's theory. This limiting value seems to be closely approached in jumps formed in water flowing at very high velocity, say, 10 ft. per sec. or greater. Assuming for the present that the actual value is the same as the limiting value, the following equations may be developed:

Neglecting friction losses, Bernouilli's theorem for the conditions shown in Fig. 29, may be written,

$$\frac{V_1^2}{2g} - \frac{V_2^2}{2g} - (d_2 - d_1) = \frac{(V_1 - k\,V_2)^2}{2g}$$

$$\frac{V_1^2}{2g} - \frac{V_2^2}{2g} - (d_2 - d_1) = \frac{V_1^2 - 2\,V_1\,k\,V_2 + k^2\,V_2^2}{2g} \quad \ldots\ldots(1)$$

Mr. Ewald.

Collecting terms:

$$\frac{2\,V_1\,k\,V_2}{2\,g} - (1 + k^2)\,\frac{V_2^2}{2\,g} = d_2 - d_1.$$

We also have the equation:

$$V_1\,d_1 = V_2\,d_2$$

and

$$\frac{d_2}{d_1} = \frac{V_1}{V_2} = \text{to } n \text{ (assumed)}.$$

Then, in the foregoing equation:

$$\frac{2\,V_1^2\,k}{2\,g\,n} - (1 + k^2)\,\frac{V_1^2}{2\,g\,n^2} = (n - 1)\,d_1$$

$$\frac{V_1^2}{2\,g}\left(\frac{2\,k}{n} - \frac{(1 + k^2)}{n^2}\right) = (n - 1)\,d_1$$

$$\frac{V_1^2}{2\,g} = \left(\frac{n - 1}{\dfrac{2\,k}{n} - \dfrac{(1 + k^2)}{n^2}}\right) d_1$$

$$\frac{V_1^2}{2\,g} = \left[\frac{n^3 - n^2}{2\,k\,n - (1 + k^2)}\right] d_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Also:

$$\frac{n^3 - n^2}{2\,k\,n - (1 + k^2)} = \frac{V_1^2}{2\,g\,d_1} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2_1)$$

In a rectangular section in which the hydraulic gradient is but little higher than the surface of the water at d_2, the velocity at d_y is approximately $\frac{3}{2}\,V_2$ and the value of k is $\frac{3}{2}$. Substituting this value in Equation (2), we get:

$$\frac{V_1^2}{2\,g} = \left(\frac{n^3 - n^2}{3\,n - 3.25}\right) d_1,$$

which is the approximate expression for the jump in open rectangular channels, provided V_2 is so small that it makes k equal $\frac{3}{2}$ nearly. Values of the expression, $\frac{n^3 - n^2}{3\,n - 3.25}$, for values of n up to 20, have been plotted as a curve in Fig. 30.

The application of this equation to a few examples of the jump will be given. One example, in which $V_1 = 14.29$; $d_1 = 0.041$; $V_2 = 1.00$; and $d_2 = 0.583$, has already been mentioned. For this case,

$$\frac{V_1^2}{2\,g\,d_1} = \frac{n^3 - n^2}{3\,n - 3.25} = \frac{3.17}{0.041} = 77.4.$$

Mr. Ewald.

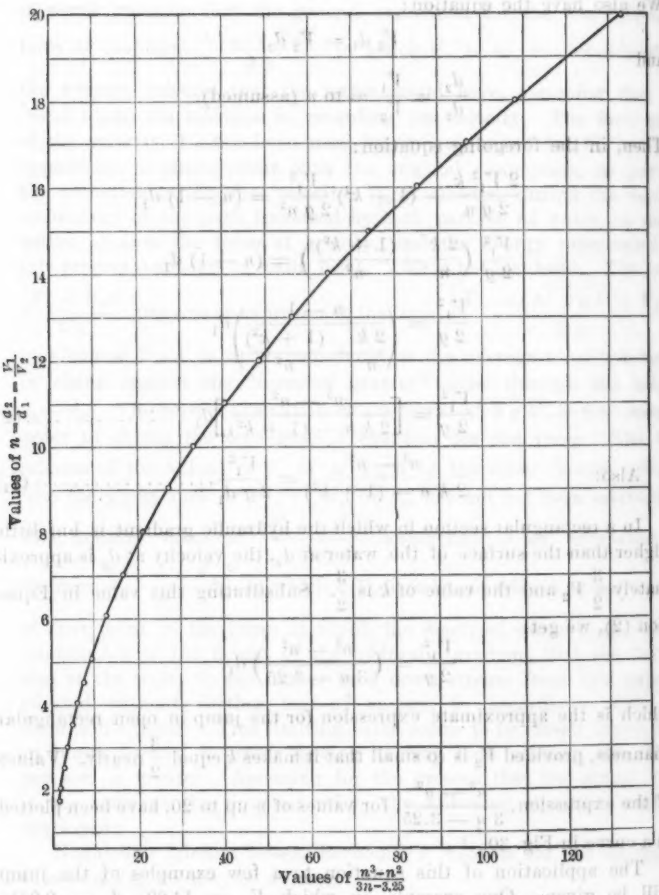

FIG. 30.

Referring to the curve, the value of n is found to be about 15.2, and nd_1 becomes $15.2 \times 0.041 = 0.624$. The observed value of d_2 was 0.583. Mr. Ewald.

The jump in the concrete chute of the Truckee Canal was not formed in a channel with a level bottom, and the equation is not exactly applicable. However, if it is assumed that the velocity of V_1 was 34 ft. for the limiting discharge of 90 sec-ft., d_1 would be 0.265, and

$$\frac{V_1^2}{2gd_1} = \frac{n^3 - n^2}{n - 3.25} = 68,$$

for which $n = 14.3$ and $nd_1 = 3.79$. The actual difference in elevation of the initial point of the jump on the slope and the surface of the water below the jump apparently is very close to 3.5 ft.

The limited data, based on Bidone's experiments, at present available to the writer, do not check the equation given, the value of n, as derived from the curve, being only 2.46, though the actual value was almost exactly 3.0 for the particular example quoted by Mr. Kennison. Bidone's experiments were made in channels having slopes of about 0.02, and the actual difference in elevation of the surface of the water at the initial point of the jump and the crest of the rise probably was considerably less than the value $(d_2 - d_1)$ used in deriving the equation. Also, as the value of V_2 was comparatively large, the value of k would differ from $\frac{3}{2}$ by a considerable amount. If the conditions of the experiments were actually as shown in Fig. 11, another reason for the discrepancy would be found in the fact that the obstruction forming the jump was nearly under the crest and deflected all the water upward toward the gradient to an extent sufficient to cause the obstruction, regardless of its small size, to act as an external controlling section, and form a rise more in the nature of the standing wave. Equation (2) will apply strictly only to those conditions in which the crest of the jump is far enough up stream to permit the water to attain a condition of uniform parallel flow before passing over the obstruction. The function of the latter is simply to maintain a depth of water sufficient to permit the minimum conditions under which internal absorption of energy can take place.

The flow of water over arched dams into channels of varying depth introduces complications necessitating individual study for each case. Observation indicates that in a dam having a uniform depth of water over the crest, the jump will be pushed farthest down stream in the shallower sections. This indicates that, instead of treating the cross-section as a whole, or averaging the depths, the entire cross-section should be divided into sections having bottoms of nearly uniform elevation, and treating each section as a rectangular channel, paying

Mr. Ewald. particular attention to the values of V_1 and d_1 for the water entering those sections.

In summing up, it appears that there are certain types of rise in open channels in which there is little or no loss of energy, and that there are other types in which the inherent energy loss is very great. The latter type is of great importance to the designer of dams, waste-ways, etc., and merits much additional study.

Mr. Cornish. L. D. CORNISH,* M. AM. SOC. C. E. (by letter).—This paper and the discussions thereon are extremely interesting, and form a valuable addition to the disseminated knowledge of hydraulics. The writer regrets that lack of time prevents him from preparing a satisfactory discussion, but the facts regarding two cases which he has observed will be submitted, and may be of interest in connection with Part 6 of the paper.

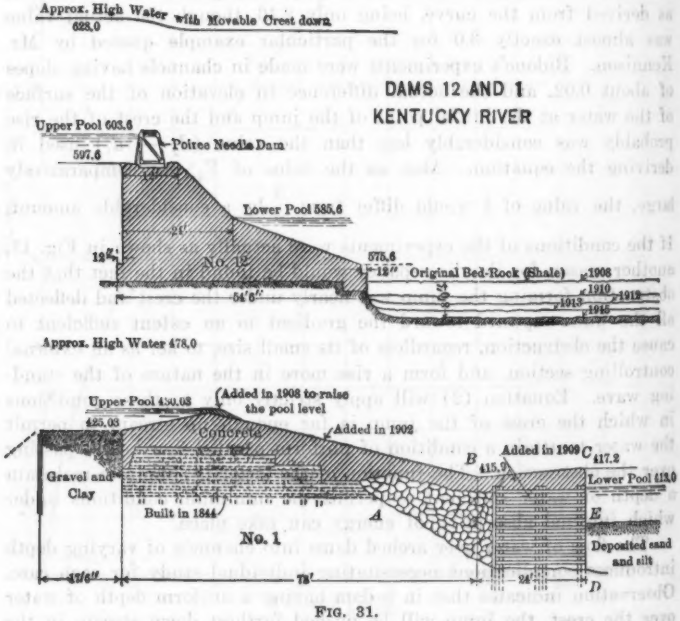

FIG. 31.

Fig. 31 shows sections of two dams which are now in operation on the Kentucky River. Dam No. 12 was put into operation in 1908, and is founded on a laminated shale which, *in situ* or in large masses,

* Cincinnati, Ohio.

Mr. Cornish.

was apparently hard and durable, although small pieces, when thoroughly dried out and subsequently immersed in water, would rapidly break up into a finely-divided condition like mud. During periods of high water the movable crest is lowered, and the water surface may be anywhere between the limits shown. As indicated by the profiles, the water is gradually eroding the shale, which had to be broken up by blasting during construction excavation, and the eroded material has formed a bar about 150 ft. below the dam. The surface of the bar consists of large pieces of shale, many of which are approximately 3 by 7 ft. and from 6 to 8 in. thick, and the entire bar has been formed by material eroded from just below the toe of the dam.

Dam No. 1, 4 miles above the point where the river empties into the Ohio, is typical of several other dams above it. It was originally a timber crib structure, built in 1844, and was capped with concrete as far as *B* in 1902. The water scoured out the river bed to a depth of 20 ft. at *D*, and undermined and carried away the crib and concrete cap as far back as *A*, for a width of 150 ft. In 1909 this damage was repaired by constructing the concrete-capped rip-rap fill and crib, since which date the erosion has ceased and fine material (*D E*) has been deposited below the crib. The approximate depth of this deposit is 12 ft., but it varies considerably during a season of frequent rises.

The absence of further erosion is attributed entirely to the apron. the inclined surface of which is typical of many movable and fixed dams on the Ohio River and its tributaries, and the construction of a similar crib-supported apron is proposed to prevent further erosion at Dam No. 12.

Mr. Saville.

Caleb Mills Saville,* M. Am. Soc. C. E. (by letter).—The hydraulic principles involved in the phenomenon of the "Hydraulic Jump" are treated in a very able and thorough manner by the author, and his paper is a most valuable and timely contribution, as so much engineering attention is being focused on "safety first" in the design of dams and appurtenant works. So clear has been the author's mathematical presentation of this subject that the most that remains is to bring forward examples of the nature of the hydraulic jump in special cases, and descriptions of the methods adopted to avoid danger in connection with this phenomenon.

In his work on the additional water supply for the City of Hartford, Conn., the writer has recently had under his direction both the design and construction of several dams, which are of considerable magnitude and importance on account of local conditions. In connection with these, it is proposed to present some features of design and observation.

* Hartford, Conn.

Mr. Saville.

The Nepaug Dam is a cyclopean masonry structure, arched in plan, and closing a narrow valley. It has both overflow and non-overflow portions, the drop from maximum reservoir level to slack water below the dam being about 107 ft. Fig. 32 shows a maximum section of the overflow portion. This portion is about 180 ft. long, and par-

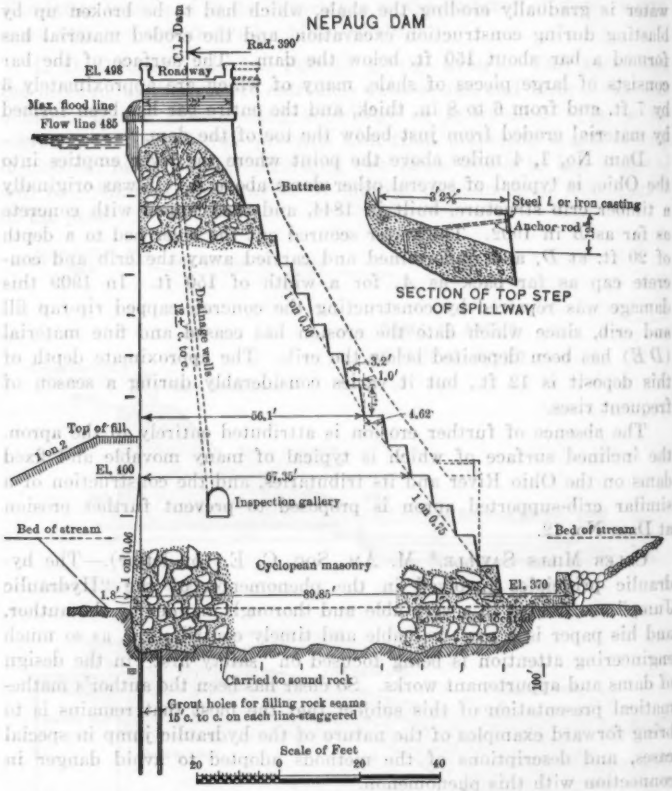

FIG. 32.

ticular attention was given to methods of breaking up the velocity which the water would acquire in falling over the dam and thus prevent it from doing damage at the down-stream base of the structure.

Two methods were used, both of which are indicated on the section. It is designed to break the smooth sheet of water passing over the ogee

crest by forcing it to jump from step to step down the incline, instead of allowing it to glide smoothly to the bottom under full momentum. As an additional help, the first step, just below the crest portion, is to be alternately depressed and raised in sections about 5 ft. long. As a second line of defence, the discharged water will be received in a pool

Mr. Saville.

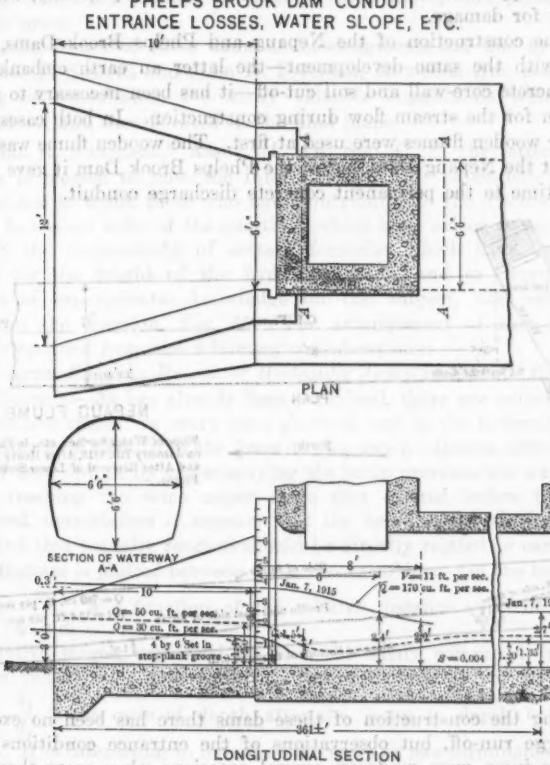

FIG. 33.

of dead water at the foot of the dam. The pool will be about 15 ft. deep, and will have a very massive concrete bottom with a baffle wall down stream. Both of these will be very heavily reinforced with steel, and the latter will be backed with large boulders. This provision is made in order to resist the force of the falling water and also the blow of any solid body that might come over the spillway. It is the expec-

Mr. Saville.

tation that the momentum of the water flowing over the spillway, on reaching the bottom, will be checked in the pool of dead water. Any energy that remains, however, will be expended, when the water is deflected back on itself by the obstruction of the baffle wall. During periods of considerable flow, the pool is expected to present a very turbulent appearance, but the water should flow away quietly, with no capacity for damage.

In the construction of the Nepaug and Phelps Brook Dams, connected with the same development—the latter an earth embankment with concrete core-wall and soil cut-off—it has been necessary to make provision for the stream flow during construction. In both cases, rectangular wooden flumes were used at first. The wooden flume was continued at the Nepaug Dam, but at the Phelps Brook Dam it gave place after a time to the permanent concrete discharge conduit.

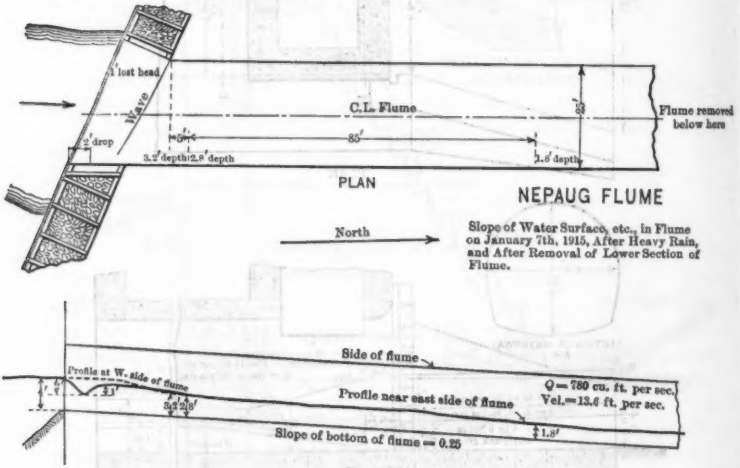

FIG. 34.

During the construction of these dams there has been no exceedingly large run-off, but observations of the entrance conditions and hydraulic jump were made on several occasions when more than the ordinary quantity of water was flowing. The conditions which obtained are shown by Figs. 33 and 34. On Fig. 33 it is interesting to note the effect on the low flows caused by the slight projection of the stop-plank. Fig. 34 shows that the construction conditions at the inlet have a very marked effect on the character of the flow at the entrance.

In the case of a third structure—Richards Corner Dam—across a stream draining 62 sq. miles of territory having very quick run-off

characteristics, a concrete conduit, having an inside width of 21 ft., and founded on ledge, has been designed to divert the water during construction. Having in mind flood conditions, when there might be some difference in head between the surface of the water in the pond and that flowing in the conduit, the apron at the approach was arched and thickened considerably in order to resist the upward pressure due to this cause. Mr. Saville.

In the design of these dams acknowledgments are due to Messrs. Frederic P. Stearns and John R. Freeman, Members, Am. Soc. C. E., acting as consultants, for many valuable suggestions which were worked out and incorporated in the final plans.

KARL R. KENNISON,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer is pleased to note the interest which this paper has aroused, and wishes to thank those who have contributed to the discussion. In order to answer some of the questions which have arisen to show more clearly the applicability of certain formulas which have been proposed for the height of the hydraulic jump, and to illustrate the extent of experimental knowledge on the subject, the writer has prepared the diagram, Fig. 35. The arrangement of this diagram was determined from the following considerations: Mr. Kennison.

Diagram Showing Height of Hydraulic Jump in Smooth Rectangular Channels.—As has already been explained, there are evidently two "alternative stages" in every open channel, and in the hydraulic jump the water level rises from the lower to the upper. Hence, although the loss of head ordinarily accompanying the jump prevents the water level from reaching the same upper stage that existed before the jump occurred, nevertheless it appears that the height theoretically attainable and that actually reached should be directly related to each other. The diagram is plotted between these two variables: On the horizontal scale, $\frac{v_1}{\sqrt{g\,d_1}}$ is a function of the relative distance apart of the two alternative stages, expressed so as to be easily computed from the known velocity, v_1, and depth, d_1, before the jump; on the vertical scale, $\frac{d_2}{d_1}$ is the ratio of depth after the jump to depth before the jump. It is interesting to note that the more or less formidable looking formulas to be found in the discussions of this paper can, without exception, be expressed with comparative simplicity in terms of these variables, and are easily plotted on this diagram.

Let the "alternative stage ratio", $\frac{v_1}{\sqrt{g\,d_1}}$, be represented by S, and the "hydraulic jump ratio", $\frac{d_2}{d_1}$, by J. Then the formula representing

* Providence, R. I.

Mr. Kennison.

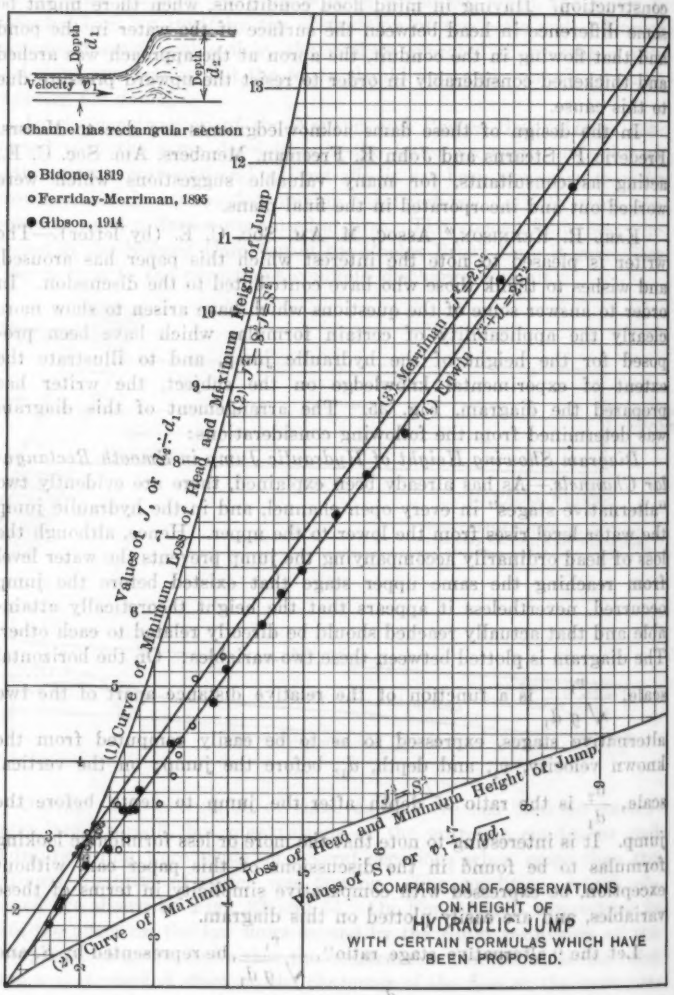

FIG. 35.

a jump to the theoretic upper alternative stage, without any loss of head whatever, is Mr. Kennison.

$$2J^2 - S^2 J = S^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

This formula, in its more complex form, was worked out in detail by Mr. Groat, on page 359, and by Mr. Johnson, on page 379. It is also identical with Professor Engel's formula, described by Mr. Andrews on page 374. In Fig. 35, this curve is labeled "Minimum Loss of Head and Maximum Height of Jump."

In direct contrast to this, the formula representing the jump to the "critical stage of maximum discharge under a given head" is

$$J^3 = S^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

In the diagram, this curve is labeled "Maximum Loss of Head and Minimum Height of Jump". Now, the only part of the diagram with which we are concerned is that lying between these two curves, as it is impossible for the jump to be higher than shown by Curve (1)* or lower than shown by Curve (2).

In Table 6 have been summarized some of the observations on the height of the hydraulic jump which have been called to the writer's attention. These have been plotted on Fig. 35. In general, these points appear to fall on a straight line drawn from the intersection of Curves (1) and (2). The vertical distance below Curve (1) corresponds to the loss of head occurring in the jump. Note that for small values of $\frac{v_1}{\sqrt{g\,d_1}}$, that is, when the two alternative stages are not far apart, the loss of head in the jump is relatively very small, as would naturally be expected. Practically all the earlier experiments, such as those of Bidone, were of this nature, and gave but little evidence as to what would happen with large values of $\frac{v_1}{\sqrt{g\,d_1}}$, as in the case of a spillway discharge. Mr. Robert Ferriday's experiments, the results of which Mr. Merriman has kindly put on record in his discussion of this paper, reached a value of 3.9 for $\frac{v_1}{\sqrt{g\,d_1}}$. Fig. 35 shows that the plotting of these results is not very uniform. One point falls quite a distance outside of Curve (1), an impossible result, which was noted also by Mr. Merriman.

The most remarkable tests are those by Professor Arnold H. Gibson referred to by Mr. Walker on page 388. One of the experiments was for a value of $\frac{v_1}{\sqrt{g\,d_1}}$ as high as 8.6. All these points come

* The actual jump could be very slightly higher since, due to our convenient erroneous assumption of uniform distribution of velocity throughout the section, the actual total energy head in the stream is slightly greater than computed.

Mr. Kennison.

remarkably close to a single line, apparently straight, and falling rapidly away from Curve (1); that is, the loss of head in the jump increases rapidly as $\frac{v_1}{\sqrt{g\,d_1}}$ increases, or as the alternative stages draw apart from each other. Some idea of what these values of $\frac{v_1}{\sqrt{g\,d_1}}$ mean is obtained by noting that, for a flood of 5 ft. over a 20-ft. dam, $\frac{v_1}{\sqrt{g\,d_1}}$ = about 7; over a 25-ft. dam, about 8; and over a 30-ft. dam, about 9. Attention is called, later, to an observation by Mr. Ewald on $\frac{v_1}{\sqrt{g\,d_1}} = 12.5$.

TABLE 6.—RATIOS, S AND J,

COMPUTED FROM OBSERVATIONS ON HYDRAULIC JUMPS.

(1) Observed mean velocity, v_1, in feet per second.	(2) Observed mean depth, d_1, in feet.	(3) Observed mean depth after jump, d_2, in feet.	(4) Alternative stage ratio, S: $v_1 \div \sqrt{g\,d_1}$.	(5) Hydraulic jump ratio, J: $d_2 \div d_1$.
BIDONE'S OBSERVATIONS; SERIES I TO IV.*				
4.47	0.155	0.423	2.00	2.73
4.45	0.155	0.437	1.99	2.82
4.41	0.156	0.430	1.97	2.76
4.52	0.152	0.436	2.04	2.87
5.59	0.208	0.613	2.16	2.94
5.56	0.210	0.620	2.14	2.95
5.52	0.211	0.630	2.12	2.98
5.49	0.212	0.642	2.10	3.02
5.67	0.206	0.647	2.20	3.14
6.29	0.246	0.738	2.24	3.00
6.36	0.244	0.745	2.27	3.05
6.39	0.242	0.764	2.29	3.15
4.55	0.150	0.398	2.07	2.65
4.57	0.150	0.405	2.08	2.70
4.57	0.150	0.428	2.08	2.85
4.59	0.149	0.423	2.10	2.84
FERRIDAY'S OBSERVATIONS; SERIES I TO VII AND IX TO XII.†				
2.18	0.050	0.143	1.72	2.86
2.98	0.044	0.150	2.50	3.41
3.56	0.036	0.153	3.31	4.25
3.66	0.033	0.168	3.55	5.10
4.39	0.095	0.267	2.51	2.81
5.02	0.083	0.285	3.07	3.44
5.02	0.071	0.290	3.32	4.08
3.50	0.055	0.162	2.63	2.95
3.95	0.046	0.175	3.25	3.80
4.06	0.042	0.188	3.50	4.48
4.33	0.038	0.205	3.92	5.40

* *Transactions*, Royal Soc. of Turin, 1819, p. 21.

† *Engineering News*, 1895, Vol. 34, p. 28; also Table 5.

TABLE 6.—(*Continued.*)

Mr. Kennison.

(1) Observed mean velocity, v_1, in feet per second.	(2) Observed mean depth, d_1, in feet.	(3) Observed mean depth after jump, d_2, in feet.	(4) Alternative stage ratio, $S: v_1 \div \sqrt{g\,d_1}$.	(5) Hydraulic jump ratio, $J: d_2 \div d_1$.
GIBSON'S OBSERVATIONS; SERIES A, B, AND C.*				
4.30	0.0735	0.265	2.79	3.61
5.82	0.0731	0.350	3.79	4.79
7.20	0.0730	0.455	4.69	6.23
8.33	0.0729	0.530	5.43	7.28
8.95	0.0728	0.570	5.85	7.83
9.74	0.0730	0.612	6.35	8.39
11.66	0.0728	0.760	7.62	10.44
13.12	0.0727	0.850	8.59	11.69
3.45	0.1465	0.267	1.59	1.82
5.68	0.1395	0.467	2.68	3.35
6.82	0.1390	0.587	3.22	4.22
8.39	0.1390	0.726	3.96	5.22
9.40	0.1390	0.808	4.44	5.81
10.53	0.1390	0.910	4.98	6.55
4.43	0.2075	0.419	1.71	2.02
4.53	0.2070	0.440	1.75	2.12
5.34	0.2048	0.530	2.08	2.59
6.09	0.2046	0.575	2.37	2.81
6.63	0.2040	0.678	2.58	3.32
7.05	0.2043	0.684	2.74	3.34
7.24	0.2043	0.735	2.82	3.59

* *Minutes of Proceedings*, Inst. C. E., Vol. CXCVII, p. 233. "The Formation of Standing Waves in an Open Stream," by Professor Arnold H. Gibson.

The diagram clearly shows the need of additional experiments with high values of $\frac{v_1}{\sqrt{g\,d_1}}$, and under a variety of conditions. The writer maintains that Professor Gibson's results, although very uniform among themselves, do not furnish sufficient evidence that the relative height of the jump will be the same under other conditions. In his experiments, which were carried on in a flume 3 ft. wide, observations were taken on the height of the tail-water necessary to bring the jump back close to a rectangular opening through which the water was being discharged along the bottom of the flume. Hence the discharge was the very steady discharge of an orifice, and not the free flow of an open channel such as Ferriday's. This may account for the remarkable uniformity of the observations.

Application of Hydraulic Jump Formulas.—In regard to the use of certain hydraulic jump formulas in the design of important structures for controlling the flow of water, Fig. 35

Mr. Kennison.

shows that the actual experiments which check these formulas are, for the most part, on small values of $\frac{v_1}{\sqrt{g\,d_1}}$, that is, under different conditions than obtain, for example, in the case of a high spillway dam. Mr. Merriman's formula, described in detail by Mr. Andrews on page 374, reduces to

$$J^2 = 2S^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

and Professor Unwin's formula, described in detail by Mr. Andrews on page 376, reduces to

$$J^2 + J = 2S^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Fig. 35 shows that the Merriman formula gives values of $\frac{d_2}{d_1}$ greater than the Unwin formula by a nearly constant quantity, about 0.5. The Merriman curve crosses Curve (1) at the point, $\frac{v_1}{\sqrt{g\,d_1}} = 2.12$; hence, for values of $\frac{v_1}{\sqrt{g\,d_1}}$ less than about 2, it will give impossibly large values of the depth, d_2; and for values of $\frac{v_1}{\sqrt{g\,d_1}}$ equal to or a little greater than this, it should give correct results, especially if the jump occurs smoothly, with very little loss of head. Note the remarkable agreement between the Merriman formula and the Bidone experiments, in all of which the value of $\frac{v_1}{\sqrt{g\,d_1}}$ was nearly constant, from 2.0 to 2.3. The Unwin formula shows a remarkable agreement with the observations of Professor Gibson. Attention has been called in the paper and the discussions to the deduction of these formulas. It is hardly necessary to discuss them further. Note that the Unwin formula, which in its deduction does not define the loss of head in the jump, gives a height of jump less than the Merriman formula, which assumes a definite loss by "sudden expansion".

Regardless of the correctness or incorrectness* of the deduction of these formulas, Fig. 35 shows that, except for the less important lower values of $\frac{v_1}{\sqrt{g\,d_1}}$, they both give results which are remarkably close to the truth, judging from the scanty information at present available. Hence, if it were a question of predicting, as closely as

* The writer wishes to emphasize the statement that there can be no doubt that the original Merriman equation is in error. Mr. Andrews, in defending it, loses sight of the fact that Bernoulli's equation takes into account the retardation in velocity due to a rise in water surface.

possible, the height of jump in a given case, the writer would not hesitate to use these formulas, with modifications to fit the variation of the actual channel from the condition of a smooth rectangular flume. However, is it not more often a question of what is the least depth which must be maintained below the discharge, in order that there may be no doubt as to the occurrence of the jump? For such a purpose, and until more observations are available to check Professor Gibson's results, the writer would use greater depths than indicated by the formulas in question. If these depths are not attainable, are not such devices as the inexpensive Bassano Dam baffle piers, or Mr. Stearns' baffle wall, proper and reasonable means for taking the occurrence of the hydraulic jump out of the realm of uncertainty? Mr. Kennison.

Alternative Stages and Alternative Slopes.—Some questions have arisen in the discussion regarding the alternative stages described in the paper. It appears that at any point, in an open channel which is carrying a certain quantity of water under a certain head, there are two and only two surface levels or alternative stages at which the water can flow steadily. This conclusion has been checked by the thorough mathematical analysis of Mr. Groat. A vertical line drawn through his diagram, Fig. 14, shows at a glance the relative position of these two stages in channels of rectangular, and also parabolic, cross-section. Although, at any particular section, the position of the alternative stages is not dependent on channel slope, neither stage can be maintained indefinitely without the proper slope of channel bed, the lower high-velocity stage naturally requiring a steeper slope than the upper low-velocity stage. Mr. Johnson has discussed at some length the relation between these slopes, very appropriately called "alternative slopes". Take, for example, the case of a stream flowing swiftly at the lower stage over a channel bed of insufficient slope. The head is gradually used up by the friction under this high velocity. As the head decreases, the two alternative stages approach each other to within striking distance, when the surface jumps to the upper stage, which the channel slope is steep enough to maintain. Such a jump, formed on a line, V-shaped in plan, due mainly to the greater depth in the center of the stream, is not an uncommon sight.

Mr. Dunham has asked, since a negative root of an equation often has an unexpected and important meaning, what is the meaning of the root — 1.82 ft. which was discarded on page 344 leaving only two depths, 9.58 ft. and 2.24 ft., corresponding to the two alternative stages. The negative root, in this case, refers merely to the lower alternative stage at a point where the channel has been depressed a distance equal to twice the depth represented by the negative root.

Mr. Kennison.

For example, Figs. 4 to 9 show the two alternative stages, 9.58 ft. deep and 2.24 ft. deep, respectively, in the case of 50 cu. ft. per sec. flowing under a total head of 10 ft. Now, referring to Fig. 5, if Pond No. 2 is removed entirely, so that the water surface drops along the dotted line, as soon as a point is reached where the channel bed is 3.64 ft. below its original level, we have depth = 1.82 ft., velocity = $\sqrt{2g \times 11.82} = 27.5$ ft. per sec.; and, because the surface of the water is — 1.82 ft. above the original channel bed, this condition happens to satisfy the cubic equation.

Loss of Head in the Jump.—Both Mr. Merriman and Mr. Ewald have had exceptional opportunities for studying the hydraulic jump. They have suggested that there are possibly two types of jumps, the "true hydraulic jump" having a smooth surface throughout, instead of the turbulent profile shown in Fig. 11. From a study of Fig. 35, this contention seems plausible. For small values of $\frac{v_1}{\sqrt{g d_1}}$, that is, when the alternative stages are close together, the observations follow very close to Curve (1); the Bidone experiments, in particular, show very little loss of head, and such jumps as these must have occurred very smoothly. Mr. Ferriday's observation—that balls of putty, carried along the bottom of the flume, stopped dead just below the toe of the jump—appears to indicate that an eddy is formed in the bottom of the channel. In such cases as Mr. Merriman's "true hydraulic jump" it is possible that the main body of water flows up over this eddy to the upper stage very smoothly, with practically no disturbance or loss of head, except that in the eddy itself. If this is true, the effect of this eddy would be similar to that of a smooth concrete weir built up from the channel bed, which may explain Mr. Ewald's reference to the jump "forming within itself the controlling section". Referring again to Fig. 35: for all velocities higher than those corresponding to the Bidone observations the water is apparently unable to maintain this smooth rise, for there is a marked falling away from Curve (1). It appears that, although there may be a difference in the appearance of two jumps under the different conditions cited, nevertheless they are fundamentally the same phenomenon, namely, the passage from the lower to the upper alternative stage. In stating that the loss of head is not an inherent part of this phenomenon, the writer had reference to the same fact that was noted by Messrs. Merriman and Ewald, namely, that in some cases the loss of head is not in evidence, although in others it is large and unavoidable. Mr. Ewald's observation of the apparent adjustment of forces in certain jumps and the steady destruction of energy with a minimum of noise, reminding him of a smoothly

running motor under full load, is particularly interesting. Additional observations may possibly show a decided drop or break in the curve of Fig. 35 when $\frac{v_1}{\sqrt{g\,d_1}}$ reaches a value of about 2.5; otherwise the conclusion would be that there is no marked difference in type between those jumps that destroy much energy and those that destroy but little. Mr. Kennison.

Mr. Ewald has developed a formula for the hydraulic jump based on an assumption of loss by "sudden expansion" from the point, d_1, to the point, d_y, in Fig. 29, and but little loss after the point, d_y, is reached. It appears that he bases this diagram and the conclusions therefrom on the suggestions of the writer in the paper. If so, these suggestions were misunderstood. The water is flowing at the lower stage at d_1 and at the upper stage at d_2; but, at intermediate points, such as d_y, the conditions of internal pressure are so disturbed that the writer would not attempt to apply the laws of open-channel flow. If Mr. Ewald had in mind a controlling section formed within the jump itself, such as referred to above, then d_y should have been measured, not from the channel bed, but from the top of this controlling section, and the depth and velocity at such a theoretic controlling section, where depth $= \frac{2}{3}$ head, would be a function solely of the quantity discharged per foot of width, and not of the depth, d_2. It seems doubtful if such a controlling section within the jump itself can be formed, except with low values of $\frac{v_1}{\sqrt{g\,d_1}}$.

Mr. Ewald quotes an interesting experiment on a model dam. The observations, reduced so as to be plotted on Fig. 35, are $\frac{d_2}{d_1} = 14.3$, and $\frac{v_1}{\sqrt{g\,d_1}} = 12.5$. Unfortunately, this point falls outside the limits of the diagram as it was originally prepared. It falls about 15% below the Unwin curve, possibly due to the fact that actual channel conditions as represented in the model caused the jump to occur at a lower depth of tail-water than would be the case in a smooth rectangular flume. It is hoped that additional observations will take note of the effect of rough channels, elevated buckets, baffle piers, etc., in reducing the height of tail-water necessary to insure a flow at the upper alternative stage.

Since his first discussion of this paper, Mr. Johnson has submitted a very interesting discussion* of the theoretical losses involved in the hydraulic jump, concluding that the Unwin formula will give the correct height of jump in smooth rectangular channels. Mr. Johnson

* Page 380.

Mr. Kennison.

has apparently shown—what had not yet been made clear—that the Unwin formula, which was developed merely by considering the momenta before and after the jump, is thoroughly consistent in its derivation with an energy equation between velocity and pressure heads before and after the jump. He has shown that the Unwin formula contains in reality allowance for a definite loss of head, equivalent, say, to the total foot-pounds per second necessary to stop the backward flow, *B*, in an eddy such as shown in Fig. 36, plus an equal amount of power necessary to carry this flow ahead again, *C*, this eddy or eddies operating under a head equal to the rise in surface in the jump, and the quantity flowing in the eddies being such that the proper relation between static head and momentum is maintained, not only at the beginning and end of the jump, as in the Unwin formula, but throughout the jump, on the basis of static pressure being proportional to the depth. The writer does not assent to this last assumption, as one would expect the conditions of internal pressure to be more or less disturbed within the jump itself. Nevertheless, the remarkable agreement of the Unwin formula with actual observations is clearly shown by Fig. 35.

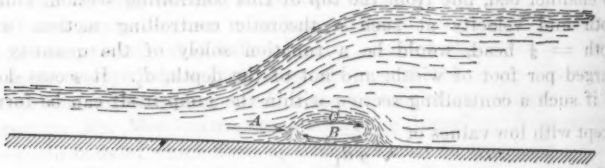

FIG. 36.

Mr. Johnson has not confined his losses to a single distinct eddy as shown at *B*, *C*, this Fig. 36 being more or less diagrammatic, to assist in discussing Mr. Johnson's theory. It also illustrates the foregoing reference to the jump forming within itself the controlling section, although, under high velocities, the loss probably occurs in many small eddies. Note that Mr. Ewald states: Pitot tube tests show that the quantity of water undergoing actual movement up stream is relatively small. Details of such tests are not given.

The total horizontal force, *A*, expended on the eddies is essential to the jump, as Mr. Johnson states, and is a measure of the loss of head in the jump; and, if this eddy was replaced by a smooth weir, no loss of head would occur, because, as Mr. Johnson clearly shows, the fastening of the weir to the channel bed keeps it from sliding up stream and obviates the necessity of the force, *A*.

Although the losses in the hydraulic jump under high velocities can be studied satisfactorily only by experimental observation, a reasonably correct theory as the basis for such studies is of great value.

Mr. Johnson's theory seems to be sounder than the few others which have been proposed, and is in excellent agreement with the actual observations now available. Mr. Kennison.

The Hydraulic Jump in Dam Design.—On account of the practical importance of the hydraulic jump in the design of spillway dams, this phase of the matter has received considerable attention in the discussions. The writer is indebted to Messrs. F. P. Stearns, C. M. Saville, L. D. Cornish, H. B. Muckleston, and E. W. Bush for the information which they have given, based on knowledge of actual conditions at important structures. Among other things, Mr. Stearns has called attention to a condition which is often overlooked, namely, the tendency of a concrete apron, from which the tail-water has been pushed down stream, to be forced upward by the excess pressure beneath it. Certain designs with which the writer has been connected have contained vent holes up through the concrete and inclined down stream, so that a slight suction would be produced to relieve the upward pressure under it.

The writer has already called attention in the paper to the destructive force of a spillway discharge. At seams in the rock foundation, the high velocity of the water is converted into pressure capable of tearing loose large masses of rock. It appears that this "Pitot tube action" is especially likely to erode rock which lies in horizontal strata. Mr. Cornish has given an excellent illustration of this in Fig. 31, and the erosion shown cannot be ascribed to the shape of the dam section. The writer has known of a similar case where horizontally stratified limestone was torn up, leaving a hole very similar to that shown below Dam No. 12 in Fig. 31. One piece of this limestone which was washed down stream weighed more than 20 tons. The erosion occurred close up to the toe of the dam, although the bucket was curved so as to deflect the discharge to an approximately horizontal direction.

The section of the Nepaug Dam shown in Fig. 32 involves noteworthy features, and the effect of the stepped face in preventing erosion of the boulder stream bed will be awaited with interest.

One important reason for knowing whether or not the flood height of tail-water is more than sufficient to cause the hydraulic jump, is this: If it is amply sufficient, then the back pressure of tail-water against the down-stream face of the dam may be counted on in computing the stability of the structure. If it is not sufficient, then none of this pressure above the bucket can be counted on, the only back pressure on the dam being the weight of the sheet of water and the reaction due to its deflection into a horizontal direction as it leaves the toe.

Mr. Kennison.

The hydraulic jump is indirectly connected with many other features in the design of works for controlling the flow of water, but the writer will not venture farther from the announced purpose of the paper, which was mainly "to call attention to the existence of the two 'alternative stages' in open-channel flow, and * * * to identify the 'hydraulic jump' as the passage from the lower to the upper stage."

In conclusion, the writer again thanks those who have contributed to the discussion, and hopes that additional observations will throw more light on the nature and extent of the energy losses occurring in the hydraulic jump.

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Paper No. 1356

THE ACTION OF WATER UNDER DAMS*

By J. B. T. Colman, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. H. B. Muckleston, Warren B. Travell, Malcolm Elliott, James B. Hays, W. P. Creager, Edward Wegmann, W. R. Baldwin-Wiseman, John C. Oakes, H. W. King, Robert E. Horton, and J. B. T. Colman.

Synopsis.

This paper represents the results of a study of the action of water under dams, and was undertaken by the writer in connection with his graduate work at the University of Michigan. The attempt is made to investigate the relation existing between the height of water on the face of a dam, the length of the base of a dam measured perpendicular to the face, the distribution and upward pressure on the base of the structure, and the effect of sheet-piling placed at the heel and at the toe of the dam.

In procuring complete data for the determination of the foregoing factors, a second field was entered: that of the relation existing between the elements just mentioned, the porosity and effective size of the sand grains, and the resulting movement of the water through the porous medium.

The conclusions resulting from this investigation are stated at the end of the paper. A few of the more important ones are as follows:

The loss of pressure head of water in passing from above to below a dam may be divided into three stages:

* Presented at the meeting of September 5th, 1915.

(*a*) A relatively large loss of head, occurring where the water enters the sand;

(*b*) A quite uniform and comparatively small loss of head per foot of length through the body of the sand;

(*c*) A larger loss of head, occurring where the water leaves the sand.

The first stage or entrance loss is increased as the length of floor is decreased.

The rate of flow decreases very rapidly with the depth below the floor of the structure, being more concentrated for shorter floors.

The flow lines, once established in a porous medium, tend to continue horizontally as such, and come to the surface only after traveling a relatively great distance.

The porosity, effective size, and uniformity coefficient have little influence on the upward pressure exerted against the floor of a dam.

Piling at the heel of a dam reduces the pressure on the floor, and piling at the toe increases it.

Piling at the heel of a dam, to be effective, must be tight, very small leakage destroying its action.

Piling at the toe of a dam should be loose, in order to prevent increasing the upward pressure on the floor of the structure.

An impervious cut-off wall of comparatively shallow depth, say 5 ft., at the heel of the structure will greatly increase its stability by reducing the upward pressure on the floor.

NOMENCLATURE.

The following notation is used in the paper:

Numbers 1 to 46, inclusive, indicate different piezometer tubes.

a, *b*, *c*, and *d* are constants.

d represents the depth of sand in the box, in feet.

H represents the head of water against the face of the dam, in feet.

h_1 represents the height of the box above the sand surface, in feet.

h_2 represents the height of the tail-water above datum, in feet.

k is a constant.

TABLE 1.—Determination of Pressure Heads in Porous Foundations.

Experiment No.	Observation No.	Date, 1914.	Hour.	Temperature, in degrees Fahrenheit.	Load on floor, in tons.	Discharge, in cubic feet per minute.	Height of Water in Tube No. 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	Remarks.
1a..	1	Jan. 19	4:15 P. M.	44.5	5	0.339	1.39	0.41	0.33	0.36	0.41	0.30	0.30	0.44	0.33	0.30	0.26	0.32	0.28	0.28	0.28	0.46	0.40	0.28	0.28	0.26	0.35	0.44	0.29	0.37	0.25	0.30	0.27	0.25	0.25	....	0.24	0.25	0.24	....	0.24	0.24	0.21	0.23	....	0.21	0.24	0.17	....	....	Sand *A*. No sheet-piling. Floor 8 ft. 3 in. long.
1..	1	" 20	9:00 A. M.	44.5	"	0.371	1.50	0.47	0.41	0.43	0.51	0.36	0.40	0.53	0.41	0.35	0.42	0.39	0.36	0.33	0.36	0.54	0.37	0.33	0.32	0.31	0.32	0.50	0.34	0.31	0.35	0.32	0.32	0.30	0.30	....	0.31	0.30	....	0.29	0.29	....	0.30	0.29	0.25	0.26	....	0.28	0.29	....	....	....	
	2		11:45 "	44.5	"	0.784	2.26	0.50	0.76	0.79	0.92	0.68	0.74	0.93	0.74	0.66	0.79	0.75	0.64	0.69	0.65	1.13	0.71	0.64	0.60	0.59	0.98	1.07	0.67	0.61	0.57	0.59	0.60	0.50	0.57	0.57	0.58	0.37	0.50	0.55	0.55	0.33	0.55	0.54	0.45	0.33	0.16	0.49	0.53	0.37	0.53	....	
2..	1	" 21	9:00 P. M.	43.0	"	1.648	3.81	2.21	1.86	1.89	2.37	1.74	1.88	3.05	1.86	1.67	2.05	1.78	1.64	1.59	1.47	2.41	1.65	1.62	1.58	1.51	1.98	2.72	1.66	1.58	1.45	1.69	1.55	1.49	1.46	1.47	1.46	....	1.28	1.26	1.49	0.77	1.39	1.42	1.11	1.33	0.96	1.22	1.34	0.84	1.31	....	
3..	1	" 22	1:45 "	44.0	"	0.436	1.17	0.96	0.41	0.52	....	0.42	0.44	0.77	0.45	0.41	0.52	0.45	0.35	0.38	0.37	0.69	0.46	0.39	0.36	0.35	....	0.66	0.40	0.31	0.34	0.37	0.36	0.34	0.35	0.35	0.33	....	0.21	0.30	0.33	....	....	....	0.34	0.31	....	0.16	0.31	0.21	0.30	....	
	2		4:00 "	44.0	"	0.944	2.39	1.53	0.61	1.14	....	0.57	0.84	1.08	1.01	0.90	1.16	0.95	0.85	0.88	0.79	1.53	1.01	0.89	0.84	0.81	....	1.49	0.91	0.84	0.80	0.95	0.83	0.78	0.80	0.79	0.80	....	0.65	....	0.78	0.42	....	....	0.60	0.72	0.19	0.41	0.74	0.50	0.71	....	
	3		8:00 "	44.0	"	1.874	4.01	3.43	1.71	2.18	....	1.54	1.90	2.86	1.92	1.76	2.11	1.92	1.69	1.68	1.59	2.72	1.80	1.70	1.60	1.57	....	2.65	1.72	1.60	1.52	1.68	1.59	1.54	1.53	1.43	1.52	....	1.26	....	1.49	0.78	....	....	1.16	1.38	0.25	....	1.39	0.90	1.36	....	
4..	1	" 24	9:15 A. M.	45.0	"	0.400	0.88	0.61	0.37	0.41	0.58	0.35	0.35	0.58	0.37	0.32	0.42	0.34	0.31	0.32	0.30	0.53	0.36	....	0.30	0.29	0.29	0.49	0.31	0.28	0.28	0.26	0.30	0.28	0.29	0.22	0.28	0.26	....	0.25	0.27	....	0.26	0.26	0.21	0.25	....	0.20	0.25	0.19	0.25	....	
	2		11:45 "	45.0	"	0.696	2.56	2.00	....	1.37	1.88	1.18	0.93	1.91	1.25	1.12	1.41	1.00	1.06	1.07	0.85	1.72	1.32	1.06	1.03	1.00	0.98	1.65	1.00	1.02	0.97	0.95	1.02	0.95	0.97	0.90	0.97	0.96	0.75	0.90	0.94	0.45	0.90	0.98	0.78	0.89	0.27	0.56	0.89	0.60	0.96	....	
	3		3:30 P. M.	45.0	"	1.760	3.48	2.99	....	2.15	2.95	1.87	1.84	2.85	1.88	1.75	2.20	1.88	1.73	1.68	1.60	2.60	1.81	1.69	1.61	1.58	1.53	2.55	1.72	1.58	1.50	1.50	1.50	1.56	1.52	1.40	1.51	1.50	1.14	1.40	1.48	0.60	1.38	1.44	1.11	1.37	0.24	0.72	1.38	0.88	1.26	....	
	4		5:45 "	45.0	"	2.786	4.86	4.11	....	3.00	4.01	2.56	2.46	3.84	2.73	2.41	3.06	2.31	2.30	2.26	1.85	3.66	2.64	2.31	2.15	2.05	2.05	3.60	2.34	2.08	1.86	2.00	2.00	2.00	1.80	1.85	1.96	0.77	1.80	1.89	1.35	1.76	0.21	0.92	1.76	0.96	1.56	....	
5..	1	" 26	8:00 A. M.	47.5	"	0.440	1.24	0.69	0.51	0.50	0.62	0.46	0.45	0.66	0.50	0.41	0.50	0.51	0.44	0.42	0.42	0.60	0.48	0.39	0.39	0.39	0.35	0.74	0.48	0.32	0.34	0.31	0.36	0.36	0.36	0.35	0.32	0.36	0.34	0.32	0.32	0.32	0.32	0.32	0.30	0.32	0.32	0.20	0.38	....	
	2		12:00 "	47.5	"	0.880	2.37	1.45	....	1.00	1.30	0.87	0.78	1.35	0.91	0.84	1.00	0.85	0.73	0.79	0.73	1.55	0.88	0.80	0.76	0.74	0.74	1.51	0.81	0.76	0.72	0.70	0.71	0.74	0.78	0.67	0.72	0.71	0.60	0.67	0.70	0.38	0.67	0.69	0.63	0.65	0.19	0.43	0.67	0.41	0.63	....	
	3		3:00 P. M.	47.5	"	1.544	3.69	2.65	....	1.88	2.48	1.63	1.22	2.42	1.68	1.59	1.87	1.45	1.34	1.50	1.33	2.66	1.66	1.53	1.44	1.40	1.38	2.59	1.51	1.48	1.43	1.55	1.49	1.56	1.53	1.51	1.40	1.50	1.48	1.37	1.40	1.46	0.97	1.38	1.48	1.07	1.34	0.39	1.14	1.34	0.80	1.30	....	
	4		7:45 "	48.0	"	1.728	3.88	2.99	1.59	2.08	2.83	1.82	1.68	2.65	1.82	1.75	2.08	1.76	1.68	1.67	1.57	2.80	1.88	1.67	1.59	1.57	1.52	2.78	1.67	1.56	1.50	1.49	1.56	1.58	1.51	1.40	1.50	1.48	1.37	1.40	1.46	0.97	1.38	1.48	1.07	1.34	0.39	1.14	1.34	0.80	1.30	....	Sand *A*. 2 ft. 9-in. piling at heel. Floor 8 ft. 3 in. long.
6..	1	Feb. 4	8:15 A. M.	49.5	"	0.544	1.20	1.01	0.72	0.79	0.96	0.65	0.73	0.96	0.58	0.60	0.82	0.52	0.57	0.56	0.53	0.97	0.47	0.52	0.51	0.50	0.54	0.49	0.45	0.49	0.48	0.48	0.36	0.46	0.46	0.48	0.47	0.35	0.46	0.48	0.30	0.41	0.45	0.18	0.43	0.46	0.31	0.42	....	0.35	0.44	0.28	0.45	....	
	2		1:30 P. M.	49.0	"	1.728	3.18	2.59	2.13	2.36	2.66	1.64	2.12	2.91	2.51	1.80	1.52	1.66	1.67	1.65	1.64	1.34	1.42	1.31	1.49	1.49	1.43	1.94	1.37	1.41	1.41	1.54	1.37	1.41	1.39	1.52	1.54	0.98	1.30	1.51	0.46	1.39	1.45	1.05	1.42	0.39	1.23	1.36	0.81	1.34	....	
	3		5:45 "	48.0	"	1.720	3.49	3.23	2.42	2.68	3.20	2.20	2.41	3.27	2.89	2.01	1.69	1.85	1.88	1.85	1.84	1.47	1.56	1.58	1.66	1.66	1.60	1.45	1.51	1.57	1.59	1.11	1.48	1.54	1.37	1.00	1.58	1.34	0.90	1.30	1.51	0.46	1.39	1.45	1.05	1.42	0.39	1.23	1.36	0.81	1.34	....	
7..	1	" 5	8:15 A. M.	50.0	"	0.607	1.20	1.04	0.79	0.87	1.08	0.75	0.78	1.07	0.92	0.67	0.56	0.92	0.64	0.61	0.61	0.49	0.53	0.50	0.55	0.51	0.54	0.48	0.51	0.52	0.52	0.36	0.50	0.52	0.52	0.35	0.51	0.52	0.52	0.36	0.50	0.52	0.52	0.35	0.51	0.52	0.52	0.36	0.50

l represents the floor length, measured on an element perpendicular to the face of the structure, and is given in feet.

M represents the moment about the heel of the dam, in foot-pounds.

o_1 represents the depth of opening below the sheet-piling at the heel, and equals $d - p_1$.

P represents the total upward pressure on 1 lin. ft. of floor, in pounds. P is used for the condition of no sheet-piling and P' for the condition of sheet-piling at the heel only.

p_1 represents the depth of sheet-piling at the heel of the structure, in feet.

p_2 represents the depth of sheet-piling at the toe of the structure, in feet.

S represents the hydraulic slope, and is equal to $\frac{H}{L}$.

x is a horizontal distance.

62.5 is the weight of 1 cu. ft. of water, in pounds.

Apparatus.

The tank or box, Fig. 1, was constructed of 2 by 12-in. by 16-ft. yellow pine, grooved on both edges for the insertion of tongues. Its inside dimensions were 15 ft. 2 in. long, 3 ft. 11¼ in. wide, and 5 ft. 6 in. high, in the lower or base portion of the dam, and 10 ft. high in the upper end, which was closed off by a bulkhead to form the height or face of the dam. The whole was held together by 4 by 6-in. yellow pine verticals, about 2 ft. from center to center, and bolted with ½-in. rods at the ends and ¾-in. rods at the centers.

Through the right side of the box and the floor of the dam extended forty-six ½-in. pipes, each of which led from a vertical plane at the center of the box through an elbow to a vertical glass tube. The tubes were adjusted to the desired height, and were connected to the pipe by rubber stoppers. The box, to a depth of 5 ft., was filled with sand which was settled by thorough wetting. On top of the sand was placed a floor, 8 ft. 3 in. long, the front or heel end of which was 4 ft. from the end of the box. On this end was placed the bulkhead, which, with the floor, formed the dam. All corners and the two sides of the floor were made water-tight by caulking the joints with oakum. The desired weight on the floor of the dam was obtained by two 18-in., 3-ton, screw jacks acting between blocking placed on the floor and the reinforced concrete ceiling.

The required head above the dam was obtained by leading water from the tap through a ½-in. hose to the chamber formed by the bulkhead. Any desired head up to 5 ft. 2 in. could be obtained by adjusting the tap. To provide for conducting waste-water from the toe of the dam, a 1¼-in. pipe was led through the side of the box to a drain. That the tail-water might be kept at a constant elevation, it was necessary to provide a means of breaking the siphon action in the pipe. This was accomplished by placing a box with a closed bottom and open top over the end of the pipe, the top of which was cut off about ⅛ in. below the desired water elevation. To aid further in regulation, several small holes were bored through the sides of the box. These were closed at times of small discharge, and the proper number opened to keep the water spilling over the top, at a uniform depth.

To assist in making observations, a straight-edge was adjusted by a level to the elevation of the tail-water surface and nailed rigidly to the 4 by 6-in. verticals of the box. This elevation was called zero. All readings are positive, and taken in feet and hundredths. Tube No. 1 gives the elevation of the free water surface. Tubes Nos. 2 to 45, inclusive, with the exception of Nos. 16, 22, 26, 30, 33, 36, and 41, pass at various points through the sides of the box, and give, in feet of water column, the equivalent pressure head in the sand at the ends of the pipes. Pipes Nos. 16, 22, 26, 30, 33, 36, 41, and 46 lead through the center of the floor and show the pressure heads against the floor of the dam directly.

Operation.

Observations were taken on two sands: *A* and *B*. The first was a natural sand, obtained from a local pit. It was of sizes varying from large pebbles to fine grains, and contained very little clay. Sand *B* was obtained by screening the first through a ½-in. square mesh screen, which removed about 35% of the larger pebbles. With Sand *A* in the box, three different sets of experiments were made: (1) with no piling at the heel or toe; (2) with water-tight piling extending to a depth of 2 ft. 7 in. at the heel; and (3) with the same piling at the heel and piling 2 ft. 3 in. deep at the toe. With Sand *B* seven sets of experiments were made: three with different lengths of floor, and four with the floor 8 ft. 3 in. long, and different depths of sheet-piling.

TABLE 2.—Determination of Pressure Heads in Porous Foundations.

Experiment No.	Observation No.	Date, 1914.	Hour.	Temperature, in degrees Fahrenheit.	Load, in tons.	Discharge, in cubic feet per minute.	Height of Water in Tube No. 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	Remarks.
17	1	Mar. 21	5:00 P. M.	48.5	5	0.544	2.40	2.31	1.10	1.27	2.27	1.00	1.06	2.16	1.68	0.95	1.01	0.93	0.92	0.91	0.91	1.18	0.89	0.88	0.87	0.86	0.88	0.81	0.77	0.81	0.74	0.67	0.69	0.76	0.76	0.67	0.69	0.70	0.36	0.43	0.68	0.30	0.54	0.59	0.27	0.63	0.19	0.41	0.53	0.11	0.45	1.30	Sand *B*. Piling at toe and 3-ft. piling at heel, floor 8 ft. 3 in. long.
	2		8:45 "	49.5	5	.....	1.90	1.62	0.89	1.01	1.80	0.80	0.85	1.72	1.33	0.76	0.80	0.74	0.74	0.74	0.77	0.69	0.69	0.70	0.69	0.67	0.62	0.61	0.66	0.64	0.52	0.55	0.60	0.61	0.49	0.55	0.56	0.29	0.34	0.51	0.23	0.43	0.47	0.21	0.41	0.08	0.33	0.41	0.08	0.37	1.05	
18	1	" 22	9:00 A. M.	50.0	5	0.384	1.96	1.87	0.94	1.06	1.85	0.86	0.90	1.75	1.36	0.82	0.82	0.79	0.80	0.79	0.79	0.82	0.72	0.74	0.75	0.75	0.72	0.65	0.64	0.70	0.69	0.57	0.58	0.66	0.67	0.52	0.60	0.62	0.31	0.40	0.56	0.30	0.43	0.53	0.24	0.47	0.16	0.36	0.47	0.09	0.41	1.05	
	2		4:15 P. M.	49.0	5	0.688	3.28	3.10	1.60	1.79	3.04	1.46	1.54	2.99	2.23	1.39	1.35	1.33	1.35	1.35	1.36	1.20	1.26	1.29	1.28	1.22	1.14	1.07	1.03	1.18	0.94	0.97	1.12	1.14	0.88	1.04	1.06	0.82	0.66	0.87	0.40	0.83	0.91	0.58	0.81	0.16	0.71	1.66	
	3		8:15 "	49.5	5	0.483	2.27	2.15	1.11	1.24	2.11	1.01	1.06	2.01	1.53	0.96	0.98	0.91	0.94	0.93	0.93	0.94	0.82	0.87	0.86	0.83	0.85	0.77	0.73	0.83	0.81	0.64	0.67	0.77	0.79	0.61	0.72	0.74	0.37	0.44	0.67	0.37	0.59	0.64	0.27	0.57	0.14	0.46	0.57	0.10	0.50	1.33	
19	1	" 23	9:20 A. M.	50.0	5	0.357	2.13	1.86	1.06	1.16	1.99	0.96	1.01	1.83	1.41	0.92	0.86	0.87	0.90	0.89	0.89	0.89	0.72	0.83	0.85	0.85	0.81	0.71	0.69	0.79	0.78	0.60	0.63	0.74	0.75	0.58	0.69	0.71	0.35	0.44	0.60	0.29	0.57	0.62	0.27	0.56	0.05	0.47	0.56	0.10	0.51	1.14	
	2		12:45 P. M.	48.0	5	0.554	3.16	2.92	1.61	1.77	2.87	1.48	1.55	2.76	2.14	1.42	1.37	1.36	1.39	1.38	1.38	1.46	1.24	1.30	1.32	1.31	1.26	1.20	1.11	1.32	1.21	0.99	1.02	1.16	1.18	0.98	1.09	1.11	0.52	1.07	1.09	0.39	0.90	0.97	0.40	0.88	0.23	0.78	0.88	0.16	0.78	1.77	
	3		2:45 "	47.0	5	1.100	4.67	4.42	2.52	2.86	4.39	2.45	2.54	4.27	3.41	2.25	2.58	2.22	2.30	2.29	2.28	2.67	2.25	2.19	2.29	2.18	2.11	2.18	1.96	2.08	2.02	1.69	1.75	1.94	1.96	1.59	1.80	1.83	0.98	1.16	1.69	0.69	1.44	1.59	0.64	1.42	0.39	1.23	1.39	0.28	1.23	2.83	
	4		5:00 "	48.0	5	0.687	3.31	3.02	1.80	1.96	3.04	1.68	1.74	2.94	2.33	1.62	1.74	1.67	1.58	1.57	1.56	1.87	1.53	1.50	1.51	1.49	1.45	1.52	1.33	1.43	1.39	1.18	1.20	1.34	1.35	1.10	1.24	1.27	0.61	0.80	1.16	0.45	1.02	1.10	0.45	1.01	0.29	0.88	1.00	0.20	0.89	2.03	
	5		8:05 "	49.0	5	0.524	2.40	2.21	1.29	1.40	2.19	1.19	1.24	2.11	1.67	1.15	1.20	1.11	1.12	1.11	1.11	1.33	1.06	1.07	1.07	1.06	1.06	1.04	0.98	1.00	0.99	0.81	0.84	0.95	0.96	0.76	0.88	0.90	0.43	0.57	0.88	0.33	0.73	0.79	0.33	0.73	0.15	0.59	0.71	0.08	0.64	1.69	
20	1	" 24	11:10 A. M.	49.5	5	0.400	2.37	2.25	1.26	1.39	2.24	1.16	1.21	2.23	1.65	1.12	1.15	1.07	1.08	1.08	1.07	1.20	1.01	1.02	1.03	1.02	0.96	0.96	0.88	0.96	0.94	0.78	0.79	0.90	0.91	0.75	0.82	0.84	0.29	0.52	0.72	0.30	0.65	0.71	0.27	0.64	0.20	0.48	0.62	0.10	0.53	1.41	Sand *B*. 2-ft. piling at heel, floor 8 ft. 3 in. long.
	2		2:15 P. M.	49.0	5	0.556	3.55	3.39	1.92	2.11	3.35	1.77	1.85	3.21	2.51	1.70	1.82	1.65	1.66	1.65	1.64	1.95	1.60	1.56	1.58	1.56	1.50	1.56	1.37	1.48	1.44	1.21	1.23	1.32	1.40	1.04	1.26	1.29	0.60	0.79	1.17	0.48	0.96	1.08	0.39	0.90	0.34	0.70	0.93	0.16	0.70	2.43	
	3		4:45 "	47.5	5	1.070	4.87	4.63	2.65	2.91	4.59	2.45	2.55	4.59	3.48	2.34	2.49	2.25	2.28	2.26	2.25	2.78	2.17	2.19	2.16	2.14	2.06	2.17	1.85	2.02	1.86	1.62	1.68	1.88	1.90	1.49	1.72	1.75	0.77	1.03	1.60	0.56	1.33	1.48	0.47	1.31	0.29	0.96	1.25	0.22	1.07	3.00	
	4		7:15 "	48.5	5	0.544	3.43	3.19	1.82	1.99	3.17	1.68	1.75	3.11	2.39	1.61	1.70	1.55	1.57	1.55	1.55	1.91	1.47	1.47	1.48	1.47	1.42	1.50	1.25	1.38	1.35	1.07	1.10	1.29	1.31	1.01	1.18	1.21	0.52	0.69	1.10	0.39	0.93	1.06	0.34	0.90	0.17	0.68	0.89	0.15	0.75	2.54	
21	1	" 25	7:45 A. M.	50.0	5	0.384	3.13	2.88	1.73	1.87	2.87	1.63	1.68	2.85	2.29	1.56	1.58	1.50	1.52	1.51	1.51	1.87	1.40	1.43	1.55	1.44	1.39	1.43	1.20	1.38	1.34	1.04	1.07	1.28	1.30	0.96	1.19	1.22	0.54	0.69	1.12	0.39	0.96	1.06	0.34	0.96	0.17	0.75	0.94	0.20	0.89	2.39	
	2		12:00 "	50.0	5	0.480	3.69	3.34	2.04	2.19	3.31	1.90	1.97	3.30	2.57	1.83	1.87	1.76	1.79	1.78	1.78	2.16	1.66	1.69	1.71	1.70	1.65	1.70	1.45	1.62	1.59	1.24	1.28	1.52	1.54	1.19	1.41	1.45	0.64	0.85	1.46	0.46	1.16	1.26	0.41	1.14	0.21	0.89	1.13	0.21	0.97	2.83	
	3		3:00 P. M.	49.5	5	0.744	4.66	4.31	2.63	2.86	4.18	2.51	2.59	4.15	3.32	2.43	2.52	2.36	2.39	2.37	2.37	2.96	2.32	2.37	2.29	2.27	2.21	2.35	2.08	2.18	2.13	1.82	1.89	2.07	2.08	1.78	1.98	1.96	0.97	1.20	1.81	....	1.55	1.71	0.57	1.56	0.25	1.23	1.53	0.29	1.23	3.53	
	4		5:15 "	50.0	5	0.495	3.57	3.21	2.05	2.19	3.19	1.92	1.98	3.17	2.52	1.87	1.91	1.81	1.84	1.82	1.82	2.24	1.78	1.78	1.75	1.75	1.70	1.81	1.61	1.68	1.64	1.47	1.44	1.59	1.61	1.35	1.41	1.52	0.66	0.90	1.23	0.43	1.13	1.33	0.45	1.22	0.23	0.98	1.20	0.26	1.06	2.73	
	5		9:30 "	50.5	5	0.480	3.57	3.22	2.06	2.21	3.20	1.95	2.02	3.18	2.55	1.90	1.96	1.84	1.86	1.85	1.85	2.28	1.80	1.78	1.79	1.77	1.73	1.82	1.63	1.71	1.67	1.50	1.47	1.62	1.63	1.39	1.52	1.55	0.69	0.97	1.44	0.52	1.26	1.36	0.46	1.26	0.28	1.00	1.23	0.26	1.07	2.70	
22	1	" 26	7:45 A. M.	51.0	5	0.356	2.91	2.65	1.76	1.88	2.66	1.68	1.72	2.65	2.14	1.63	1.66	1.58	1.60	1.59	1.59	1.92	1.53	1.52	1.53	1.52	1.48	1.50	1.36	1.43	1.39	1.27	1.25	1.39	1.40	1.18	1.31	1.33	0.69	0.91	1.26	0.40	1.09	1.18	0.40	1.08	0.20	0.87	1.07	0.19	0.98	2.39	
	2		10:45 "	50.5	5	0.352	2.80	2.61	1.83	1.41	1.99	1.25	1.28	1.99	1.60	1.31	1.18	1.17	1.19	1.18	1.18	1.38	1.09	1.13	1.14	1.14	1.11	1.07	0.96	1.09	1.07	0.91	0.93	1.08	1.05	0.85	0.96	1.00	0.50	0.81	0.95	0.36	0.82	0.89	0.50	0.81	0.15	0.67	0.80	0.15	0.71	1.66	
	3		3:10 P. M.	51.5	5	0.294	1.76	1.58	1.01	1.08	1.92	0.95	0.98	1.81	1.20	0.92	0.98	0.89	0.91	0.90	0.90	0.90	0.83	0.86	0.87	0.87	0.84	0.78	0.75	0.83	0.82	0.68	0.68	0.79	0.80	0.66	0.76	0.76	0.32	0.46	0.71	0.29	0.63	0.68	0.24	0.63	0.13	0.51	0.63	0.10	0.55	1.94	
	4		7:15 "	52.0	5	0.177	1.43	1.24	0.82	0.87	1.22	0.76	0.79	1.22	0.97	0.75	0.71	0.72	0.74	0.73	0.73	0.76	0.67	0.69	0.70	0.70	0.68	0.66	0.61	0.67	0.66	0.55	0.55	0.66	0.64	0.54	0.60	0.62	0.28	0.36	0.57	0.25	0.51	0.55	0.21	0.50	0.13	0.41	0.50	0.10	0.45	0.99	
23	1	" 27	7:45 A. M.	52.5	5	0.192	1.35	1.16	0.79	0.84	1.15	0.75	0.76	1.14	0.98	0.73	0.71	0.71	0.71	0.71	0.71	0.77	0.66	0.68	0.69	0.68	0.67	0.67	0.62	0.61	0.66	0.66	0.55	0.68	0.64	0.54	0.60	0.61	0.29	0.33	0.53	0.25	0.47	0.51	0.21	0.46	0.12	0.37	0.47	0.09	0.42	0.92	
24	1	" 28	7:30 "	51.0	5	0.400	1.81	1.75	1.07	1.17	1.75	1.01	1.06	1.75	1.47	1.00	1.22	1.12	0.99	0.96	0.96	1.36	1.03	0.98	0.94	0.91	0.89	0.98	0.91	0.90	0.85	0.81	0.81	0.83	0.89	0.75	0.76	0.77	0.38	0.50	0.70	0.32	0.59	0.65	0.28	0.56	0.16	0.45	0.55	0.12	0.47	1.51	Sand *B*. 2-ft. piling at heel, floor 8 ft. 3 in. long.
25	1	" 29	9:00 "	51.0	5	0.368	1.89	1.83	1.22	1.32	1.82	1.16	1.21	1.82	1.55	1.14	1.29	1.24	1.13	1.10	1.09	1.41	1.18	1.11	1.06	1.05	1.03	1.03	1.01	1.04	0.96	0.90	0.90	0.96	0.96	0.84	0.88	0.89	0.45	0.59	0.81	0.33	0.83	0.75	0.31	0.67	0.19	0.49	0.65	0.12	0.55	1.57	
	2		1:50 P. M.	50.5	5	0.304	2.26	2.19	1.46	1.58	2.18	1.38	1.45	2.16	1.89	1.36	1.59	1.46	1.35	1.32	1.31	1.67	1.37	1.32	1.29	1.26	1.22	1.24	1.18	1.23	1.18	1.06	1.06	1.14	1.15	1.00	1.06	1.08	0.52	0.66	0.96	0.39	0.82	0.91	0.38	0.80	0.21	0.65	0.79	0.17	0.68	1.80	
26	1	" 30	8:30 A. M.	51.0	5	0.400	2.40	2.30	1.61	1.72	2.29	1.53	1.58	2.24	2.01	1.50	1.66	1.53	1.49	1.46	1.45	1.78	1.46	1.45	1.40	1.37	1.35	1.27	1.36	1.32	1.17	1.14	1.26	1.29	1.07	1.19	1.21	0.57	0.76	1.13	0.34	0.93	1.07	0.48	0.95	0.21	0.80	0.94	0.22	0.93	1.93	
	2		1:30 P. M.	51.0	5	0.432	2.61	2.50	1.74	1.86	2.47	1.65	1.72	2.42	2.18	1.63	1.78	1.70	1.61	1.58	1.58	1.86	1.57	1.57	1.52	1.52	1.52	1.49	1.38	1.47	1.43	1.25	1.24	1.39	1.40	1.17	1.29	1.32	0.67	0.88	1.22	0.52	1.05	1.15	0.47	1.04	0.23	0.85	1.03	0.21	0.89	2.03	
	3		4:40 "	50.0	5	0.365	4.63	4.07	3.03	3.32	4.07	2.81	2.94	4.04	4.26	2.71	2.86	2.78	2.67	2.63	2.62	2.99	2.41	2.53	2.55	2.52	2.44	2.16	1.96	2.36	2.35	1.49	1.67	2.22	2.27	1.45	2.05	2.15	0.81	1.14	1.96	0.64	1.66	1.86	0.68	1.67	0.30	1.39	1.65	0.34	1.43	3.20	
	4		8:40 "	51.0	5	0.330	1.93	1.85	1.24	1.34	1.85	1.15	1.21	1.83	1.67	1.12	1.19	1.16	1.11	1.09	1.09	1.26	1.00	1.05	1.05	1.04	1.01	0.90	0.80	0.98	0.97	0.59	0.68	0.91	0.94	0.59	0.85	0.89	0.34	0.47	0.82	0.30	0.69	0.78	0.30	0.70	0.18	0.58	0.65	0.15	0.60	1.38	
27	1	" 31	8:30 A. M.	48.0	5	0.276	1.83	1.74	1.19	1.29	1.74	1.12	1.17	1.73	1.59	1.09	1.15	1.12	1.08	1.06	1.06	1.24	0.97	1.03	1.03	1.00	0.99	0.80	0.78	0.96	0.95	0.59	0.67	0.90	0.90	0.59	0.84	0.88	0.35	0.47	0.81	0.31	0.69	0.78	0.30	0.67	0.17	0.50	0.66	0.15	0.61	1.35	
	2		11:45 "	48.5	5	0.512	3.69	3.58	2.49	2.69	3.58	2.36	2.46	3.57	3.31	2.31	2.47	2.37	2.29	2.25	2.25	2.57	2.19	2.21	2.20	2.17	2.12	2.08	1.91	2.09	2.05	1.69	1.71	1.97	2.00	1.60	1.84	1.89	0.85	1.15	1.76	0.64	1.51	1.67	0.67	1.50	0.35	1.23	1.48	0.32	1.30	2.68	
	3		1:45 P. M.	50.0	5	0.465	3.66	3.35	2.50	2.69	3.54	2.36	2.45	3.53	3.28	2.30	2.44	2.36	2.28	2.25	2.24	2.58	2.17	2.20	2.20	2.16	2.11	1.97	1.89	2.08	2.04	1.69	1.66	1.96	1.99	1.57	1.82	1.89	0.84	1.18	1.74	0.61	1.50	1.66	0.66	1.50	0.35	1.23	1.48	0.32	1.30	2.66	
	4		6:45 "	50.5	5	1.005	4.85	4.66	3.22	3.48	4.66	3.04	3.15	4.62	4.29	2.94	2.96	2.99	2.91	2.88	2.86	3.11	2.61	2.78	2.80	2.73	2.69	2.38	2.23	2.62	2.60	1.76	1.91	2.43	2.53	1.72	2.31	2.40	0.98	1.30	2.21	0.69	1.90	2.11	0.77	1.90	0.37	1.56	1.95	0.31	1.63	3.98	
28	1	April 1	7:45 A. M.	49.5	5	0.368	2.71	2.66	1.88	2.00	2.59	1.78	1.84	2.55	2.35	1.71	1.75	1.77	1.72	1.71	1.70	1.99	1.51	1.55	1.56	1.54	1.51	1.37	1.32	1.57	1.55	1.09	1.25	1.49	1.52	1.05	1.40	1.45	0.59	0.79	1.35	0.48	1.16	1.29	0.51	1.16	0.29	0.96	1.15	0.23	1.01	2.04	
29	1	" 2	2:30 P. M.	50.5	5	0.425	2.46	2.36	1.67	1.69	2.31	1.48	1.53	2.35	1.72	1.45	1.66	1.51	1.43	1.41	1.41	1.26	1.27	1.38	1.28	1.36	1.32	1.08	1.05	1.30	1.29	0.88	0.92	1.22	1.26	0.82	1.15	1.19	0.47	0.62	1.10	0.40	0.94	1.05	0.39	0.94	0.27	0.75	0.93	0.16	0.80	1.97	Sand *B*. 1-ft. piling at heel, floor 8 ft. 3 in. long.
	2		8:45 "	51.0	5	0.544	2.44	2.37	2.90	2.38	2.18	2.08	2.12	2.32	2.10	2.00	1.97	1.97	1.95	1.73	1.92	1.93	1.90	1.85	1.42	1.43	1.80	1.79	1.18	1.24	1.70	1.74	1.12	1.50	1.66	0.64	0.93	1.52	0.54	1.30	1.46	0.51	1.31	0.29	1.03	1.30	0.22	1.12	1.70			
30	1	" 3	7:45 A. M.	48.5	5	0.504	3.63	3.49	2.46	2.63	3.43	2.33	2.39	3.48	2.64	2.21	2.64	2.35	2.26	2.22	2.22	2.67	1.94	2.17	2.18	2.14	2.10	1.57	1.60	2.04	2.03	1.30	1.39	1.93	1.98	1.97	1.35	1.80	1.86	0.70	1.68	1.97	0.74	0.58	1.49	1.66	0.55	1.49	0.32	1.20	1.48	0.25	1.27	1.88	
	2		12:00 "	49.0	5	0.690	4.84	4.66	3.30	3.53	4.62	3.11	3.20	4.68	3.56	3.08	3.54	3.14	3.00	2.97	2.95	3.54	2.53	2.69	2.90	2.85	2.78	2.00	2.07	2.70	2.70	1.65	1.57	2.77	2.55	2.61	1.85	2.36	2.50	0.97	1.18	2.30	0.69	1.95	2.19	0.99	1.96	0.37	1.37	1.94	0.33	1.68	2.33	
	3		1:30 P. M.	50.0	5	0.753	4.85	4.71	3.35	3.57	4.64	3.16	3.25	4.70	3.60	3.08	3.58	3.19	3.05	3.03	3.00	3.38	2.58	2.94	2.95	2.90	2.82	2.04	2.11	2.75	2.75	1.66	1.80	2.60	2.67	1.80	2.44	2.54	0.88	1.30	2.35	0.69	1.90	2.24	0.70	2.00	0.37	1.61	1.96	0.36	1.71	2.63	
	4		4:45 "	50.5	5	0.448	3.58	3.48	2.51	2.67	3.44	2.38	2.44	3.48	2.69	2.32	2.70	2.40	2.30	2.26	2.27	2.76	1.92	2.22	2.22	2.20	2.15	1.51	1.56	2.09	2.09	1.24	1.34	1.68	1.90	1.19	1.61	1.96	0.68	0.90	1.79	0.54	1.53	1.71	0.54	1.55	0.31	1.24	1.53	0.25	1.31	1.79	
	5		8:30 "	51.0	5	0.572	3.67	2.97	2.17	2.29	2.93	2.05	2.11	2.97	2.31	2.01	2.34	2.09	1.99	1.97	1.97	1.92	1.65	1.92	1.93	1.90	1.86	1.48	1.33	1.80	1.81	1.05	1.15	1.72	1.76	1.08	1.61	1.68	0.59	0.79	1.56	0.50	1.34	1.49	0.50	1.35	0.29	1.09	1.34	0.23	1.16	1.54	
31	1	" 4	7:45 A. M.	49.0	5	0.415	2.48	2.36	1.76	1.85	2.34	1.68	1.71	2.35	1.87	1.54	1.90	1.69	1.68	1.61	1.61	1.99	1.35	1.58	1.59	1.56	1.53	1.29	1.15	1.49	1.49	0.92	0.97	1.42	1.45	0.86	1.38	1.32	0.55	0.69	1.29	0.45	1.12	1.24	0.45	1.13	0.22	0.97	1.31				Sand *B*. 1-ft. piling at heel.
	2		1:30 P. M.	50.5	5	0.112	1.71	1.62	1.24	1.28	1.61	1.17	1.19	1.61	1.29	1.15	1.32	1.18	1.14	1.13	1.13	0.86	0.95	1.10	1.10	1.09	1.08	0.77	0.79	1.04	1.04	0.66	0.70	0.90	1.01	0.64	0.94	0.97	0.42	0.51	0.90	....	0.79	0.87	0.34	0.79	0.26	0.65	0.79	0.17	0.69	0.89	
	3		5:45 "	51.5	5	0.368	2.13	2.00	1.48	1.58	1.97	1.43	1.47	1.98	1.60	1.40	1.68	1.46	1.40	1.38	1.38	1.72	1.46	1.45	1.46	1.34	1.34	1.31	0.97	0.98	1.21	0.89	0.88	0.97	1.22	0.85	1.15	1.20	0.60	0.82	1.12	0.42	0.97	1.08	0.41	0.97	0.29	0.78	0.97	0.20	0.84	1.14	
32	1	" 6	1:45 "	50.5	5	0.304	1.80	1.69	1.23	1.34	1.65	1.21	1.24	1.66	1.35	1.19	1.45	1.23	1.18	1.17	1.17	1.42	1.17	1.16	1.16	1.14	1.12	1.18	0.96	1.11	0.99	0.86	0.87	1.05	1.07	0.81	0.97	1.01	0.45	0.66	0.98	0.39	0.79	0.88	0.34	0.80	0.28	0.60	0.78	0.11	0.65	1.48	Sand *B*. No piling at heel or toe, floor 8 ft. 3 in. long.
	2		5:30 "	50.5	5	0.416	2.89	2.70	1.87	2.09	2.61	1.68	1.83	2.72	2.20	1.62	1.81	1.80	1.78	1.76	1.78	2.16	1.72	1.78	1.75	1.71	1.70	1.43	1.69	1.66	1.65	1.25	1.34	1.59	1.62	1.16	1.48	1.55	0.62	0.80	1.42	0.50	1.31	1.49	0.46	1.32	0.31	1.03	1.30	0.20	1.01	2.17	
	3		8:45 "	50.5	5	0.416	3.11	2.90	2.14	2.26	2.81	2.09	2.08	2.54	2.24	1.99	2.39	2.05	1.97	1.95	1.96	2.29	1.93	1.92	1.93	1.88	1.85	1.78	1.59	1.81	1.79	1.30	1.31	1.71	1.74	1.20	1.60	1.67	0.65	0.84	1.58	0.58	1.29	1.46	0.48	1.31	0.31	1.01	1.30	0.22	1.09	2.31	
33	1	" 7	9:15 A. M.	47.0	5	0.345	2.98	2.79	2.12	2.28	2.73

The three floor lengths were: 8 ft. 3 in., 6 ft. 3 in., and 4 ft. 3 in. The observations with the sheet-piling were, with piling 1, 2, and 3 ft. deep at the heel, and with piling 3 ft. deep at the heel and 18 in. deep at the toe.

A set of experiments is made up of two or more experiments, each of which comprises a number of different readings. After water was turned on, the time allowed for the head to adjust itself, before making a reading, was not less than 2 hours. This was done in order that the pressure head might be transmitted to all points in the tank and that the resulting tube readings would indicate the proper relation of one tube to the other.

The pressure on the floor of the dam was kept as nearly uniform as possible throughout the experiments. This was done by frequently adjusting the jacks to a definite pull of a spring balance on the end of a 1-ft. lever arm. To determine the proper pull, the jacks, with the balance, were first gauged or rated in one of the laboratory testing machines. This pressure, which amounted to 5 tons in all except the last two sets of experiments, was uniformly distributed over a floor area 3 ft. 11 in. wide and 8 ft. 0 in. long (3 in. of the floor under the bulkhead was not connected to the main floor except by caulking) and amounted to about 320 lb. per sq. ft. The loading was intended to approximate what might be expected from a vertical-faced, reinforced concrete structure of a similar height.

In the last two (or short floor length) experiments, the load was 3.75 tons for the 6-ft. length and 2.5 tons for the 4-ft. length, which, as before, gives an average pressure of 320 lb. per sq. ft. With the 4 ft. 3-in. floor length two additional observations were taken, one with the load increased to 5 tons, or 640 lb. per sq. ft., and one with a load of 1 ton, or about 125 lb. per sq. ft.

In order to determine the quantity of seepage, the discharge, for a time varying from 1 to 5 min., was turned into a tank which rested on a scale. This was weighed and recorded for each observation.

Data.

For convenience and simplicity all the data have been arranged in the form of tables and diagrams.* Tables 1 and 2 (Plates XIV and XV), are of similar form, the first being for Sand *A* and the

* All these diagrams have been filed in the Library of the Society for reference.

second for Sand *B*. The experiment and observation numbers have no special significance; they merely indicate the method of keeping the readings in order. In general, the experiment number refers to the day on which readings were taken, and the observation number refers to the different readings taken in a single day. The date and hour of making the observation are given, and also the temperature of the water, in degrees Fahrenheit, the approximate load on the floor, in tons, and the seepage through the sand beneath the dam, in cubic feet per minute. The remarks column indicates the arrangement of the box while making each set of experiments. The columns having the general heading, "Height of Water in Tube No. ——" give the individual readings of elevation of water in the glass tubes above tail-water, as previously described.

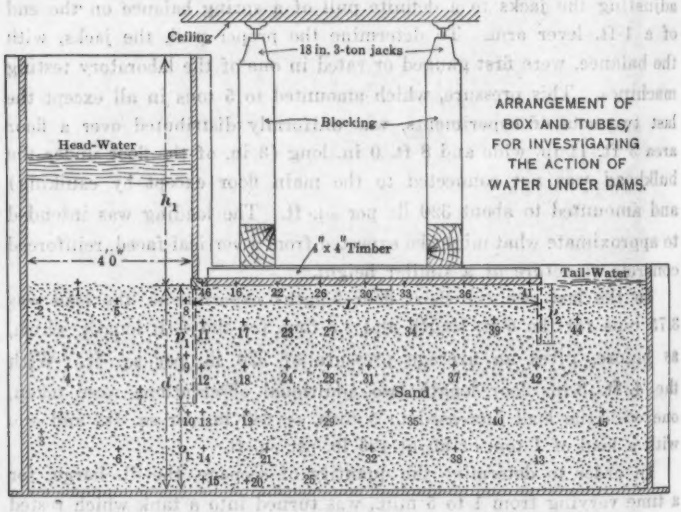

FIG. 1.

Table 3 with Fig. 1 forms a key to the succeeding curves, and sums up the ten sets of observation, as previously outlined, indicating the porosity, effective size, and uniformity coefficient of the sands, and the dimensions, as shown by the letters on Fig. 1. Fig. 1 is a section of the box on the center line, and shows the arrangement of the tubes and their numbers.

TABLE 3.—Sand Analysis, Box Arrangement, Measurements, Etc.

Experiment Nos.	Sand.	Porosity.	Effective size.	Uniformity coefficient.	Remarks.	Height of head-water chamber, h_1	Height of tail-water, h_2	Depth of sand, d	Depth of piling at heel, p_1	$(d - p_1)$ o_1	Depth of piling at toe, p_2	Length of floor, L
1–5	*A*	0.39	0.04	8	No sheet-piling	5 ft. 2 in.	0 ft. 0 in.	5 ft. 0 in.	0 ft. 2¾ in.	4 ft. 9¼ in.	0 in. 0 ft.	8 ft. 3 in.
6–11	*A*	0.39	0.04	8	Sheet-piling at heel	5 " 2 "	0 " 0 "	5 " 0 "	2 " 7 "	2 " 3 "	0 " 0 "	8 " 3 "
12–16	*A*	0.39	0.04	8	Sheet-piling at heel and toe	5 " 2 "	0 " 0 "	5 " 0 "	2 " 7 "	2 " 3 "	2 " 3 "	8 " 3 "
17–19	*B*	0.33	0.027	5.4	Sheet-piling at heel and toe	5 " 0 "	0 " 0 "	5 " 0 "	3 " 0 "	2 " 0 "	1 " 6 "	8 " 3 "
20–23	*B*	0.33	0.027	5.4	Sheet-piling at heel	5 " 0 "	0 " 0 "	5 " 0 "	3 " 0 "	2 " 0 "	0 " 0 "	8 " 3 "
24–28	*B*	0.33	0.027	5.4	Sheet-piling at heel	5 " 0 "	0 " 0 "	5 " 0 "	2 " 0 "	3 " 0 "	0 " 0 "	8 " 3 "
29–31	*B*	0.33	0.027	5.4	Sheet-piling at heel	5 " 0 "	0 " 0 "	5 " 0 "	1 " 0 "	4 " 0 "	0 " 0 "	8 " 3 "
32–34	*B*	0.33	0.027	5.4	No sheet-piling	5 " 0 "	0 " 0 "	5 " 0 "	0 " 2¾ "	4 " 9¼ "	0 " 0 "	8 " 3 "
35–37	*B*	0.33	0.027	5.4	No sheet-piling	5 " 0 "	0 " 0 "	5 " 0 "	0 " 2¾ "	4 " 9¼ "	0 " 0 "	8 " 3 "
38–39	*B*	0.33	0.027	5.4	No sheet-piling	5 " 0 "	0 " 0 "	5 " 0 "	0 " 2¾ "	4 " 9¼ "	0 " 0 "	8 " 3 "

Sand Analysis.

That the data might not be limited in their use, but have a general application, the porosity and size of the sand grains of samples of each sand were carefully determined. Five samples of Sand *A* had porosities of 0.284, 0.294, 0.300, 0.289, and 0.284, respectively, the average being 29 per cent. Four samples of Sand *B* had porosities of 0.319, 0.319, 0.318, and 0.319, respectively, or an average porosity of approximately 32 per cent.

For determining the size of the sand grains, it was realized that the method must be reasonably accurate, easy of duplication, and preferably serviceable in the field. Accordingly, the sieve analysis method was adopted. This consists in using a series of sieves, putting a sample of known weight in the coarsest, shaking the sieves until no more sand would pass, and then weighing the portion of the sample passing each sieve.

The effective size (that is, a size such that 10% by weight is smaller and 90% is larger) of Sand *A* was found to be 0.04 in., and of Sand *B* 0.027 in. The uniformity coefficient, or the ratio of the 90% to the 10%, is 8 for Sand *A* and 5.4 for Sand *B*. The sieve size and weights passing each are shown in Table 4.

TABLE 4.—Analyses of Sand.

Sieve No.	Sand *A*.				Sand *B*.			
	Sample A.		Sample B.		Sample C.		Sample D.	
	Weight passing, in pounds.	Percentage of 10-lb. sample.	Weight passing, in pounds.	Percentage of 10-lb. sample.	Weight passing, in pounds.	Percentage of 10-lb. sample.	Weight passing, in pounds.	Percentage of 10-lb. sample.
200	.....	.....	.....	.....	0.05	0.5	.....	.....
150	0.06	0.06	0.06	0.6	.....	.....	.....	.....
100	.....	.....	.....	.....	0.25	2.5	0.24	2.4
80	0.14	1.4	0.11	1.1	.....	.....	.....	.....
60	.....	.....	.....	.....	0.54	5.4	.....	.....
50	0.30	3.0	0.29	2.9	.....	.....	0.36	3.6
20	1.38	13.8	1.45	14.5	2.32	23.2	2.39	23.9
12	2.31	23.1	2.30	23.0	3.74	37.4	3.87	38.7
8	.....	.....	.....	.....	5.28	52.8	.....	.....
6	4.24	42.4	4.00	40.0	.....	.....	6.67	66.7
4	5.55	55.5	5.13	51.3	7.94	79.4	8.27	82.7
2	7.79	77.9	7.11	71.1	9.98	99.8	9.96	99.6
1	9.31	93.1	8.46	84.6	.....	.....	.....	.....
Sample	10.00	100.0	10.00	100.0	10.00	100.0	10.00	100.0
Effective size	= 0.04				= 0.027			
Uniformity coefficient	= 8				= 5.4			

Plotting.

All readings were plotted in the form of equal-pressure curves. Briefly, this consists of preparing an outline of the box, with the location of the tubes indicated thereon by crosses and numbers. The scale adopted was 1 in. = 1 ft., both vertical and horizontal. Having prepared the outline, the pressure head, as read from the tubes, was marked on each, and the points of equal pressure were connected by contours. These curves at once give a picture of what takes place in the porous sand medium beneath the structure.

For the purpose of leveling the data, all observations were also plotted in the form of head, pressure-head curves. That is, the heads against the dam were plotted as ordinates and the pressure heads in the tubes as abscissas. Individual points were plotted for each tube reading, different marks being used for each arrangement of the box. All points were plotted before drawing any curves. A whole family of points, or set of experiments, was then studied, and the form of curve which would best fit all points was determined. The curves were then drawn so as to conform closely to the plotted points, and at the same time not diverge from the general family type. These curves served a double purpose, first to study the data and see how the pressure varied from point to point, and second to level the data, that is, to charge one error in observation against another and obtain the average or more probably correct reading.

A few representative equal-pressure curves for the different arrangements of the box with Sand *B*, are shown on Figs. 2 to 8. The *H*, with each equal-pressure diagram, refers to the head against the dam when the observation was made. The experiment number, observation number, and data refer to the data in Table 2 (Plate XV).

With readings taken from the leveling curves for the tubes numbered 46, 16, 22, 26, 30, 33, 36, and 41, ten series of pressure curves were constructed. These represented the amount and distribution of the upward pressure on the floor of the dam for each set of experiments. Each series of pressure curves was made up of five curves, one for each foot of head from 1 to 5 ft., inclusive. The scale used originally was 1 in. on the paper = 1 ft. of pressure head; so that if the total area below the pressure curve was found to be *A* square inches, the upward pressure, in pounds on a 1-ft. length of dam, would

be 62.5 $\times$ A. The seven floor-pressure curves for the various arrangements of the box with Sand B are shown in Figs. 9 to 15.

The next step was to measure, with a planimeter, the areas below each of the curves and determine the distance from the heel of the dam to the centroid or center of pressure for each area. These are arranged in convenient form in Table 5. Using the slopes (or the head divided by the floor length) as ordinates and the areas as abscissas, the areas were then plotted in ten slope-pressure curves, Figs. 16 and 17.

INVESTIGATORS.

Before proceeding to the discussion of the data, it might be interesting to review briefly the findings of the all too few investigators of this important subject—"The Action of Ground-Waters." In 1856, Henri Darcy,* stated that the velocity of flow of ground-water was proportional to the slope of the hydraulic gradient, and varied inversely as the length of the porous medium through which it acted. Isaac Roberts,† demonstrated that the flow of water through sandstone varies with the pressure. Seelheim,‡ of Wiesbaden, stated that the velocity of flow increases faster than the pressure producing it.

Allen Hazen, M. Am. Soc. C. E., states§ that the flow varies approximately as the square of the effective size of the sand grains. J. H. Fuertes, M. Am. Soc. C. E., also shows‖ that the velocity coefficient is materially affected by variation in the size of the sand grains. R. Cohen** proves that the viscosity of water varies inversely as the pressure. Several experimenters have shown that the viscosity of water increases with the temperature.

W. R. Baldwin-Wiseman,†† experimenting on the porosity of rock, shows that the flow increases faster than the pressure, also that the variation of pressure throughout a column is greatest near the surface, though the variation within the mass is more uniform and relatively small.

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* "Les Fontaines Publiques de la Ville de Dijon."
† *Proceedings*, Liverpool Geological Society, 1868–69.
‡ *Zeitschrift für Analytische Chemie*, Vol. 19 (1880).
§ Report, Massachusetts State Board of Health, 1892.
‖ *Engineering Record*, December 10th, 1898.
** *Annalen der Physik und Chemie*, Leipzig, 1892.
†† *Minutes of Proceedings*, Inst. C. E., Vol. CLXV (1905–1906).

C. S. Schlichter* gives a mathematical proof and formula for the flow of ground-water, which is:

$$Q = 0.2012 \frac{p\, d^2\, s}{\mu\, h\, k},$$

in which Q is the number of cubic feet of water transmitted by a column of sand in 1 min.; p is the difference in pressure at the ends of the column under which the flow takes place, in feet; d is the effective size of the sand grains, in millimeters; μ is the number which takes account of the friction between the particles of water, or the coefficient of viscosity; and k is a constant which depends on the porosity of the sand.

Mr. Allen Hazen, who experimented with the vertical flow in sand filters, gives the following formula for the velocity of water through sand:

$$v = c\, d^2 \frac{h}{l} (0.70 + 0.03\, t),$$

in which v is the velocity of water, in meters, daily, in a solid column of the same area as that of the sand; c is a constant approximating 1 000; d is the effective size of the sand grains; h is the loss of head between the two ends of the sand column; l is the thickness through which the water passes; and t is the temperature, on the centigrade scale.

Other formulas might be given, but they would only be repetitions, and would not serve to clarify the subject. It is obvious that some of these equations are mathematical solutions of assumptions, and that others are derived from uniformly distributed vertical flow. The true laws of ground-water action can be shown only by experiment with horizontal flow; and, because of their complex nature and the variation of the factors, the work must be conducted on a large scale.

Discussion.

The principles governing the action of water under pressure in the sand forming the foundation of any structure, such as a weir or dam, may be compared with the action of water in a large number of small pipes. The upper ones would represent the sand layer adjacent to the weir floor, the second ones the next lower layer, and so on, until, in a sand of relatively great depth, the tubes representing the

* Nineteenth Annual Report, U. S. Geological Survey, Part II.

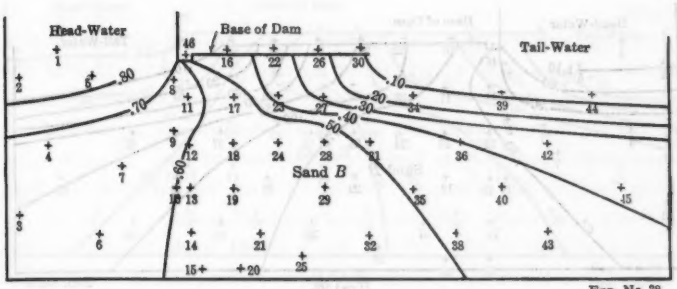
lower layers would have a length many times that of those in the first layer. These imaginary tubes extend from different points in the floor of the reservoir to points at various distances down stream from the toe of the weir.

Lines of Equal Pressure.

In taking up the discussion of the equal-pressure contours, dams without sheet-piling at the heel or toe will first be considered. All lines of equal pressure start from the lower surface of the floor of the dam and extend in directions, varying from nearly horizontal up stream to a similar position down stream from the structure. These lines naturally fall into three divisions: The first, or higher-pressure contours are relatively closer together, and, in carrying out the pipe analogy, would be comparable with entrance pressure. The difference in pressure between any two contours represents loss of head, and is comparable with entrance loss in pipes. The second division comprises the contours through the body of the sand. They are quite uniformly spaced, and are comparable with the loss of head in pipes due to friction. The third division is that of the lower-pressure contours, which are closer to one another and indicate a greater loss of head in the last portion. Thus it is seen that, in any element of water which passes from above the dam to a position below the structure, there is, through entrance eddies in the first division, a comparatively great loss of head. After this, the flow lines become nearly parallel, and head is expended at a uniform rate until near the vicinity of exit from the sand particles. Here eddies again break up the flow lines and tend to resist the exit of the water from the sand.

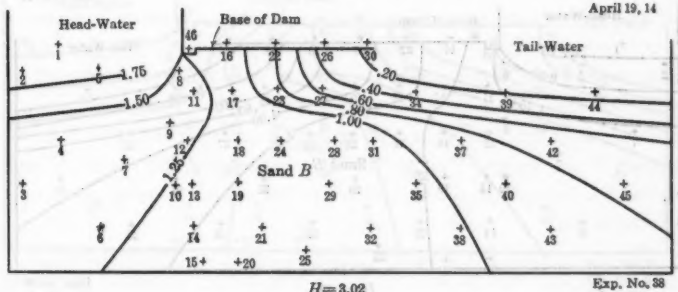
Lines drawn on each diagram perpendicular to the pressure contours would indicate the paths followed by the particles of water from point to point, and show the distribution of flow. These lines would be much closer to one another just below the floor of the dam than at greater depths. This indicates that the flow is greatest near the floor and decreases with the depth.

By comparing the pressure diagrams for different lengths of floor, it will be seen that the shorter floors give the greater entrance loss. It will also be noticed that the flow lines in the shorter floor diagrams are concentrated near the floor more than in the longer ones. In



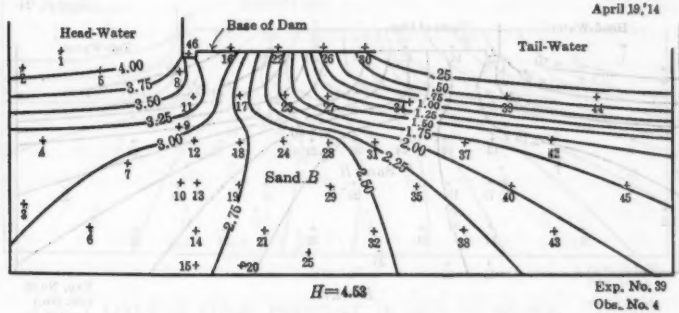
$H = 1.69$

Exp. No. 38
Obs. No. 1
April 19, 14



$H = 3.02$

Exp. No. 38
Obs. No. 4
April 19, '14



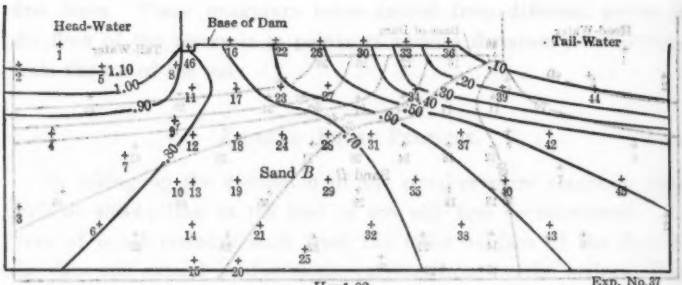
$H = 4.53$

Exp. No. 39
Obs. No. 4
April 20, '14

LINES OF EQUAL PRESSURE, IN FEET OF WATER, UNDER MODEL DAMS.

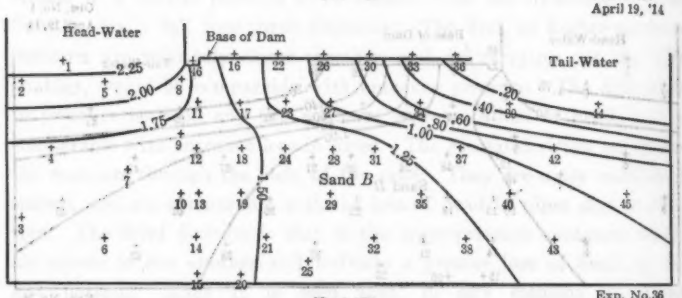
Numbers represent piezometer tubes
See Table 2 for data

FIG. 2.



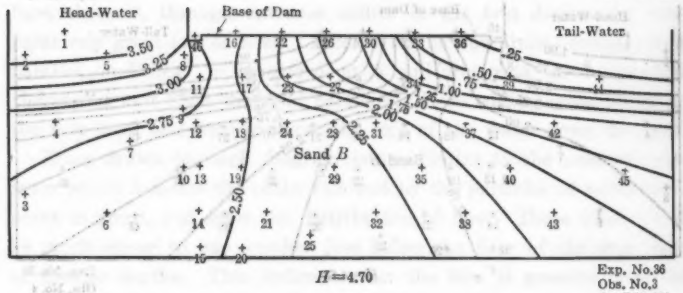
$H = 1.92$

Exp. No.37
Obs. No.2
April 19, '14



$H = 3.33$

Exp. No.36
Obs. No.5
April 18, '14



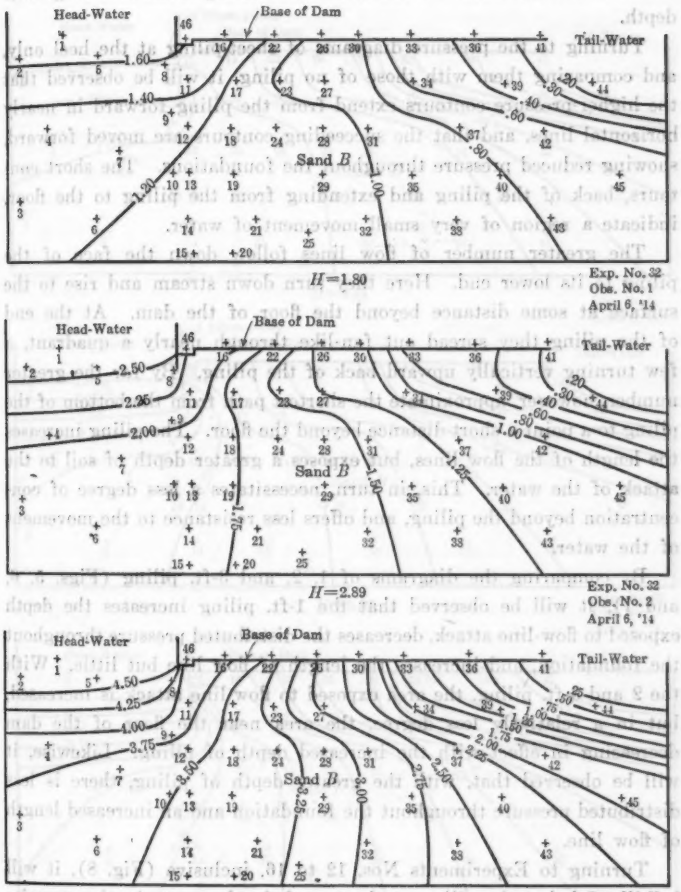
$H = 4.70$

Exp. No.36
Obs. No.3
April 18, '14

LINES OF EQUAL PRESSURE, IN FEET OF WATER, UNDER MODEL DAMS

Numbers represent piezometer tubes
See Table 2 for data

FIG. 3.



LINES OF EQUAL PRESSURE IN FEET OF WATER, UNDER MODEL DAMS.

Numbers represent piezometer tubes
See Table 2 for data

FIG. 4.

this case there are but few lines, and a resulting small flow at greater depth.

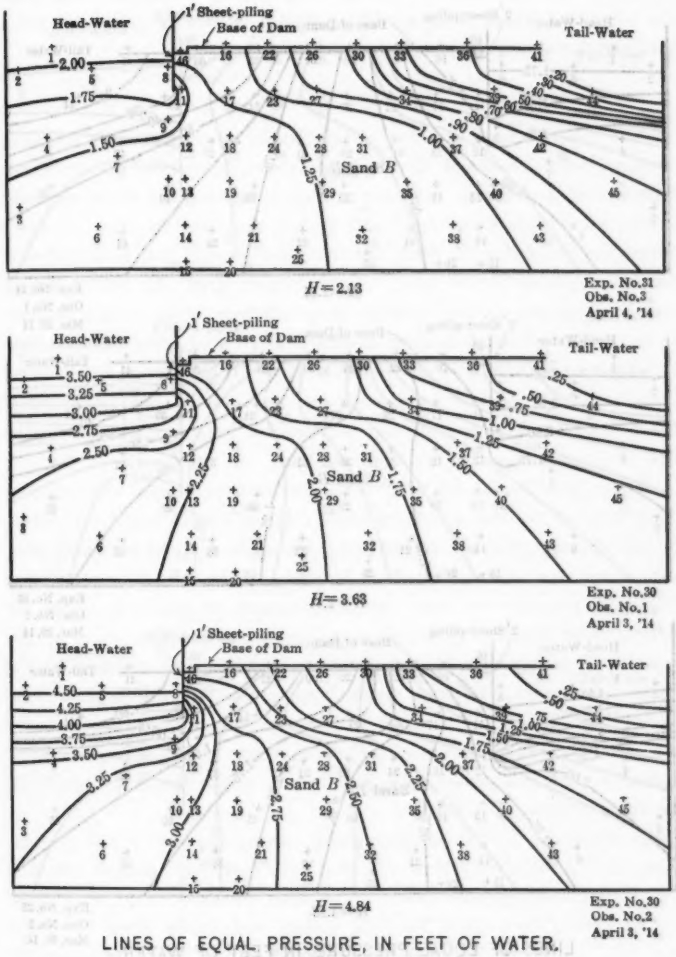
Turning to the pressure diagrams of sheet-piling at the heel only, and comparing them with those of no piling, it will be observed that the higher-pressure contours extend from the piling forward in nearly horizontal lines, and that the succeeding contours are moved forward, showing reduced pressure throughout the foundations. The short contours, back of the piling and extending from the piling to the floor, indicate a region of very small movement of water.

The greater number of flow lines follow down the face of the piling to its lower end. Here they turn down stream and rise to the surface at some distance beyond the floor of the dam. At the end of the piling they spread out fan-like through nearly a quadrant, a few turning vertically upward back of the piling. By far the greater number, however, approximate the shortest path from the bottom of the piling to a point a short distance beyond the floor. The piling increases the length of the flow lines, but exposes a greater depth of soil to the attack of the water. This, in turn, necessitates a less degree of concentration beyond the piling, and offers less resistance to the movement of the water.

By comparing the diagrams of 1, 2, and 3-ft. piling (Figs. 5, 6, and 7), it will be observed that the 1-ft. piling increases the depth exposed to flow-line attack, decreases the distributed pressure throughout the foundation, and increases the length of flow line but little. With the 2 and 3-ft. piling, the area exposed to flow-line attack is increased, but in a relatively less degree, the area near the floor of the dam decreasing in effect with the increased depth of piling. Likewise, it will be observed that, with the greater depth of piling, there is less distributed pressure throughout the foundation and an increased length of flow line.

Turning to Experiments Nos. 12 to 16, inclusive (Fig. 8), it will be observed that the piling at the toe of the dam acts in the opposite manner to that at the heel. This decreases the area exposed to the flow-line attack, and increases the pressure throughout the foundation; but, acting in a way similar to the heel piling, it also increases the length of flow line.

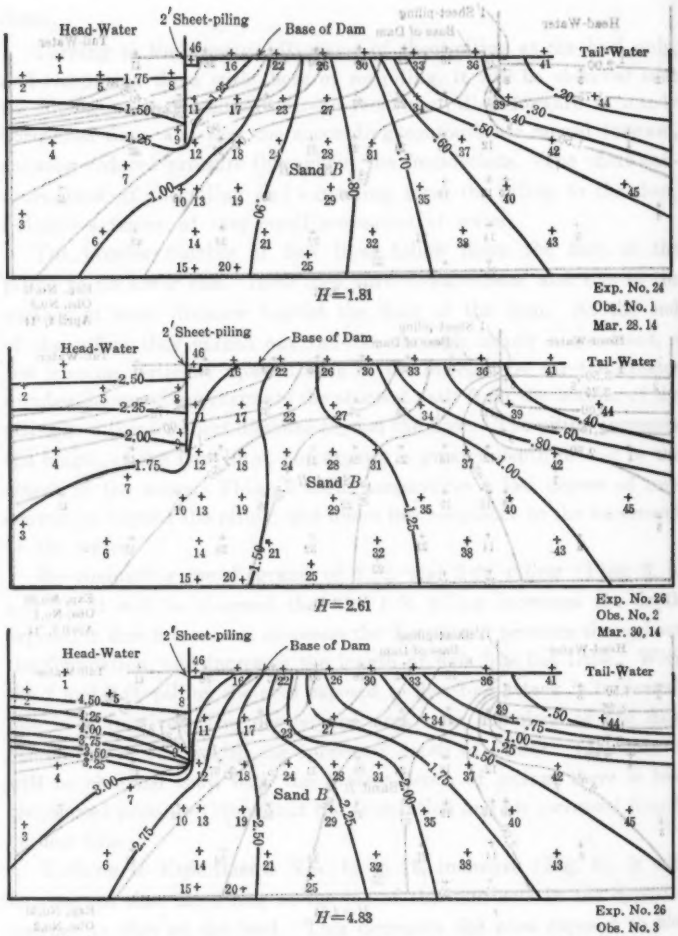
To make the piling at the heel effective, it must be water-tight. It was found that a very small leakage through the piling destroyed



LINES OF EQUAL PRESSURE, IN FEET OF WATER, UNDER MODEL DAMS

Numbers represent piezometer tubes
See Table 2 for data

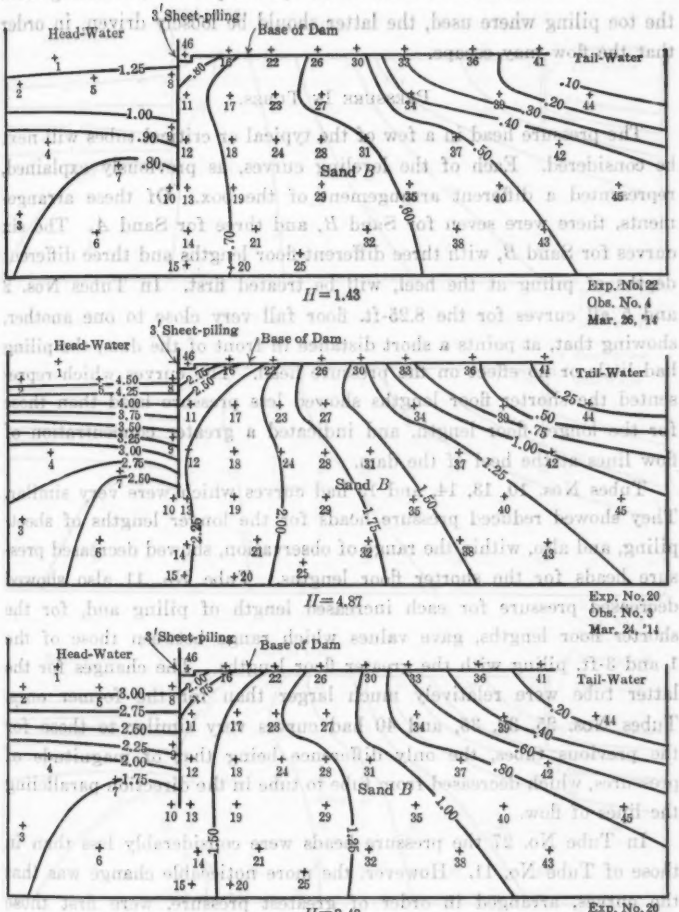
FIG. 5.



LINES OF EQUAL PRESSURE, IN FEET OF WATER, UNDER MODEL DAMS.

Numbers represent piezometer tubes
See Table 2 for data

FIG. 6.



LINES OF EQUAL PRESSURE, IN FEET OF WATER, UNDER MODEL DAMS.

Numbers represent piezometer tubes
See Table 2 for data

FIG. 7.

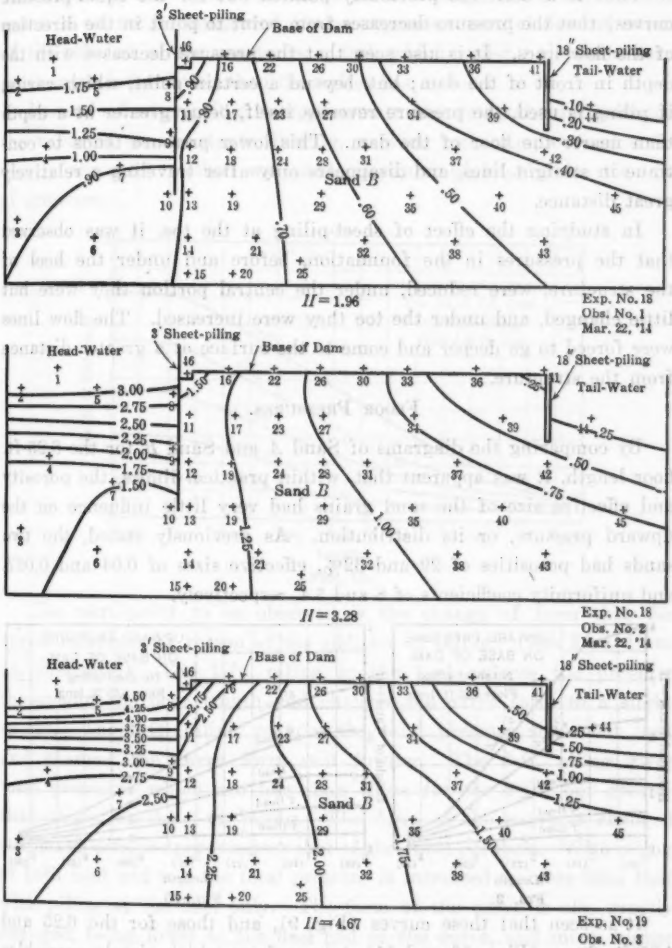
its effect. To eliminate the increased upward pressure, resulting from the toe piling where used, the latter should be loosely driven, in order that the flow may escape.

Pressure In Tubes.

The pressure head in a few of the typical or critical tubes will next be considered. Each of the leveling curves, as previously explained, represented a different arrangement of the box. Of these arrangements, there were seven for Sand *B*, and three for Sand *A*. The six curves for Sand *B*, with three different floor lengths and three different depths of piling at the heel, will be treated first. In Tubes Nos. 2 and 5 all curves for the 8.25-ft. floor fall very close to one another, showing that, at points a short distance in front of the dam, the piling had little or no effect on the pressure head. The curves which represented the shorter floor lengths showed less pressure head than those for the longer floor length, and indicated a greater concentration of flow lines at the heel of the dam.

Tubes Nos. 10, 13, 14, and 15 had curves which were very similar. They showed reduced pressure heads for the longer lengths of sheet-piling, and also, within the range of observation, showed decreased pressure heads for the shorter floor lengths. Tube No. 11 also showed decreased pressure for each increased length of piling and, for the shorter floor lengths, gave values which range between those of the 1 and 3-ft. piling with the greater floor lengths. The changes for the latter tube were relatively much larger than for the former ones. Tubes Nos. 25, 32, 35, and 40 had curves very similar to those for the previous tubes, the only difference being that of magnitude of pressures, which decreased from tube to tube in the direction paralleling the lines of flow.

In Tube No. 27 the pressure heads were considerably less than in those of Tube No. 11. However, the more noticeable change was that the curves, arranged in order of greatest pressure, were first those for no piling, then those for 2-ft. piling, and finally those for the 1 and 3-ft. piling, which were almost identical. This is explained by the action of the flow lines. The greater number for the 2-ft. piling passed close to Tube No. 27, whereas, for the 1 and 3-ft. piling, they passed above or below this point. It is also seen that, for the shorter floor lengths, the pressure was greatly reduced at this point.



LINES OF EQUAL PRESSURE, IN FEET OF WATER, UNDER MODEL DAMS.

Numbers represent piezometer tubes
See Table 2 for data

FIG. 8.

Thus it is seen (as previously pointed out for the equal-pressure curves) that the pressure decreases from point to point in the direction of the flow lines. It is also seen that the pressure decreases with the depth in front of the dam; but, beyond a certain point, which varies, if piling is used, the pressure reverses itself, being greater at a depth than nearer the floor of the dam. This lower pressure tends to continue in straight lines, and disappears only after traveling a relatively great distance.

In studying the effect of sheet-piling at the toe, it was observed that the pressures in the foundation, before and under the heel of the structure, were reduced, under the central portion they were but little changed, and under the toe they were increased. The flow lines were forced to go deeper and come to the surface at a greater distance from the structure.

Floor Pressures.

By comparing the diagrams of Sand *A* and Sand *B* for the 8.25-ft. floor length, it was apparent that, within practical limits, the porosity and effective size of the sand grains had very little influence on the upward pressure, or its distribution. As previously stated, the two sands had porosities of 29 and 32%, effective sizes of 0.04 and 0.027, and uniformity coefficients of 8 and 5.4, respectively.

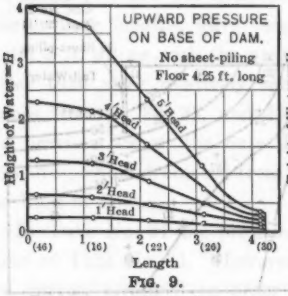


Fig. 9.

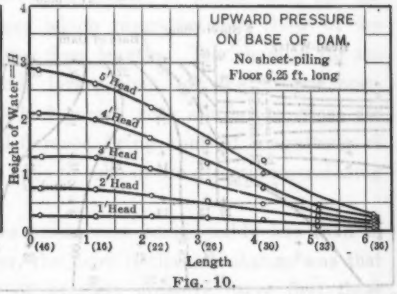


Fig. 10.

It is seen that these curves (Fig. 9), and those for the 6.25 and 4.25-ft. floors (Figs. 10 and 11), take a form which closely resembles the curve of the law of error. In fact, if P represents the pressure at any point, and x the distance from the heel to P, the law is very closely approximated by the equation:

$$P = a\,10^{-kx^n}$$

where a, — k, and n are functions of the floor length and head. As this equation cannot be integrated, its use requires a careful determination of the relation of the constants, a task which would necessitate very much more information than the writer has at hand.

Instead of attempting to approximate the integral, or determine the value of the constants directly, a much more simple (and sufficiently accurate) method was used, as explained later under the law of pressures.

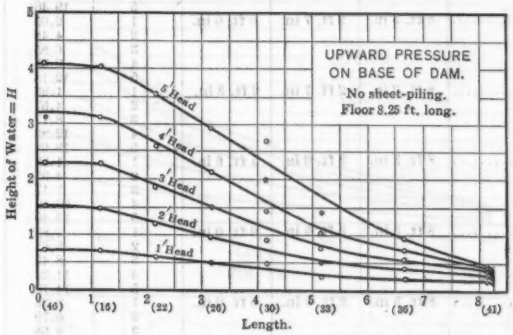


FIG. 11.

The next point to be observed is the change of form that the curve undergoes by reason of the addition of piling. (See Figs. 12 to 15, inclusive.) The 1-ft. piling at the heel reduces the pressure throughout the floor length and changes the curve form to a slight extent. The 2-ft. piling reduces the total pressure somewhat less, and changes the curve form still further. The 3-ft. piling gives total pressures which are less than those for the 2-ft., but greater than those for the 1-ft. piling. The form of the curve is changed still more until it approximates that of the cubic parabola. With piling at both heel and toe, the total pressure is increased to more than that with piling at the heel only. The form of the curve is also greatly changed, being lower at the heel end of the curve, and much higher at the toe end.

The areas of these pressure curves, and also the distances from the heel of the dam to the center of pressure are given in Table 5. By studying the values of the centroid distances, it will be seen that, for all floor lengths without piling, the center of pressure approximates

TABLE 5.—AREAS OF UPWARD-PRESSURE CURVES, AND DISTANCES OF THE CENTROIDS FROM THE HEEL OF THE DAM.

Sand.	Floor length.	Piling.		H, in feet.	Area of pressure curve, in square feet.	Distance from heel of dam to centroid, in feet.
		Heel.	Toe.			
A.	8 ft. 3 in.	0 ft. 0 in.	0 ft. 0 in.	1	2.51	2.45
				2	5.92	2.50
				3	9.98	2.45
				4	14.60	2.48
				5	19.46	2.55
A.	8 ft. 3 in.	2 ft. 7 in.	0 ft. 0 in.	1	2.05	2.60
				2	4.48	2.86
				3	6.85	2.80
				4	9.38	2.75
				5	12.18	2.70
A.	8 ft. 3 in.	2 ft. 7 in.	2 ft. 3 in.	1	1.93	3.40
				2	4.57	3.69
				3	8.17	3.67
				4	12.83	3.77
				5	18.22	3.82
B.	8 ft. 3 in.	3 ft. 0 in.	1 ft. 6 in.	1	1.96	2.63
				2	4.35	2.62
				3	7.17	2.60
				4	10.12	2.52
				5	13.40	2.58
B.	8 ft. 3 in.	3 ft. 0 in.	0 ft. 0 in.	1	2.83	2.66
				2	5.63	2.45
				3	8.42	2.38
				4	11.22	2.38
				5	14.10	2.37
B.	8 ft. 3 in.	2 ft. 0 in.	0 ft. 0 in.	1	3.35	2.46
				2	6.70	2.45
				3	9.68	2.39
				4	12.97	2.38
				5	13.97	2.34
B.	8 ft. 3 in.	1 ft. 0 in.	0 ft. 0 in.	1	2.77	2.78
				2	5.30	2.72
				3	7.78	2.74
				4	10.07	2.74
				5	12.45	2.73
B.	8 ft. 3 in.	0 ft. 0 in.	0 ft. 0 in.	1	3.44	2.50
				2	6.80	2.45
				3	10.61	2.40
				4	14.61	2.51
				5	19.38	2.50
B.	6 ft. 3 in.	0 ft. 0 in.	0 ft. 0 in.	1	1.25	2.25
				2	2.90	2.00
				3	5.19	2.00
				4	7.63	2.06
				5	10.11	1.91
B.	4 ft. 3 in.	0 ft. 0 in.	0 ft. 0 in.	1	0.78	1.55
				2	1.93	1.55
				3	3.47	1.36
				4	6.02	1.36
				5	9.70	1.28

very closely to one-third of the floor length from the heel $\left(\frac{L}{3}\right)$. For piling at the heel, this value is decreased slightly for the 1-ft., and increased slightly for the 2 and 3-ft. piling; but the error introduced by using the arbitrary value, $\frac{L}{3}$, would be very small, and on the

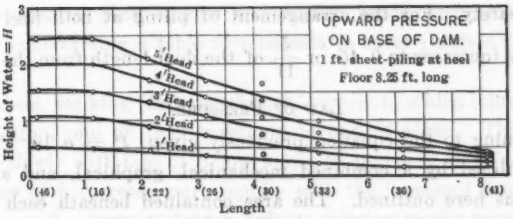


FIG. 12.

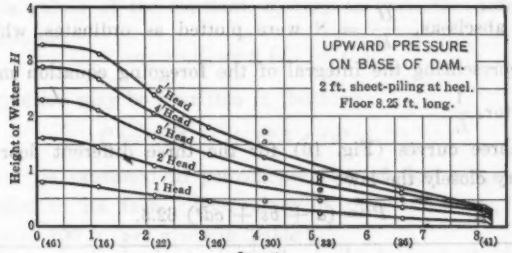


FIG. 13.

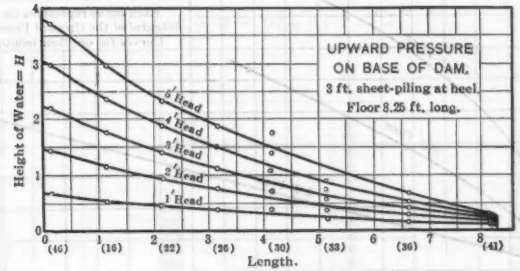


FIG. 14.

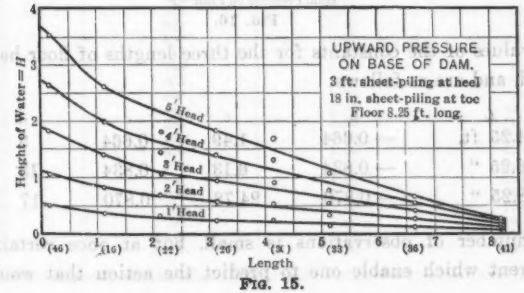


FIG. 15.

side of safety. For the arrangement of piling at both heel and toe, this value increases to 0.45, or $\frac{5}{11}$ of the floor length from the heel.

Law of Pressure.

Returning to the equation previously given, $P = a\ 10^{-kz^n}$. This was simplified by a combined mechanical, graphical, and analytical method, as here outlined. The area contained beneath each pressure curve was carefully measured with a planimeter. Then, on these areas as abscissas, $\frac{H}{L} = S$ were plotted as ordinates, which gave curves representing the integral of the foregoing equation multiplied by the factor, $\frac{1}{L}$.

The three curves (Fig. 16) for the three different floor lengths follow very closely the law:

$$P = (a + bs + cd^s)\ 62.5.$$

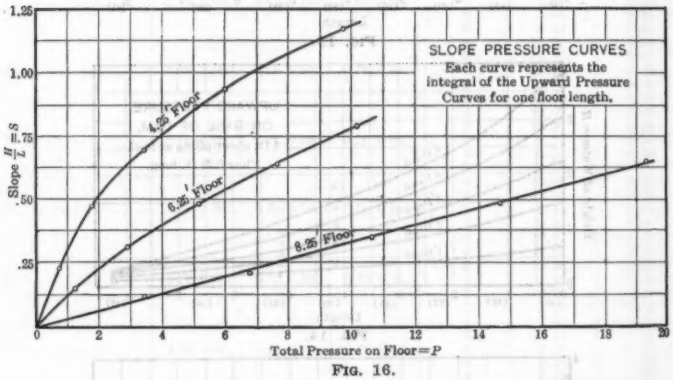


FIG. 16.

The values of the constants for the three lengths of floor have been computed, and are as follows:

L	a	b	c	d
4.25 ft.	— 0.664	1.49	0.664	9
6.25 "	— 0.834	6.13	0.834	13
8.25 "	— 0.870	24.78	0.870	17

The number of observations is small, but at once certain facts are apparent which enable one to predict the action that would take

place for greater floor lengths and higher heads, and provides the means of constructing a table of constants to cover possible cases that may arise.

The most striking fact is that $a = -c$, which changes the equation from one of four constants to one of three. Their value increases slowly with that of L, and seems to approach unity as a limit. The values of b increase more rapidly with the increased floor length, but, like a and c, seem to bear a definite relation to the length of the floor. The values of the constant, d, vary as twice the difference in floor length, or:

$$d_1 - d_0 = 2\ (L_1 - L_0).$$

This equation may be rewritten in the form:

$$P = 62.5\ bs + 62.5\ c\ (d^s - 1),$$

in which P is expressed in pounds per foot of dam measured on a line parallel to its face. The values of these constants, for floor lengths up to 100 ft., are given in Table 6.

TABLE 6.—CONSTANTS FOR THE FORMULA:

$$P = 62.5\ bs + 62.5\ c\ (d^s - 1).$$

L	b	c	d
4	1.3	0.60	8.5
6	5.1	0.82	12.5
8	24	0.86	16.5
10	48	0.88	20.5
12	80	0.90	24.5
14	120	0.91	28.5
16	165	0.92	32.5
18	222	0.93	36.5
20	278	0.94	40.5
22	234	0.95	44.5
24	393	0.96	48.5
26	450	0.96	52.5
28	508	0.97	56
30	565	0.97	60
32	623	0.97	64
34	680	0.97	68
36	737	0.98	72
38	795	0.98	76
40	853	0.98	80
45	997	0.99	90
50	1 142	0.99	100
55	1 286	0.99	110
60	1 431	0.99	120
65	1 576	1.00	130
70	1 720	1.00	140
75	1 865	1.00	150
80	2 009	1.00	160
85	2 154	1.00	170
90	2 298	1.00	180
95	2 443	1.00	190
100	2 587	1.00	200

In applying this law, it should be remembered that the center of pressure of the force, P, is $\frac{L}{3}$ from the heel of the dam, and if moments are taken about that point, M, in foot-pounds, is indicated by the expression:

$$M = \frac{P\,L}{3}.$$

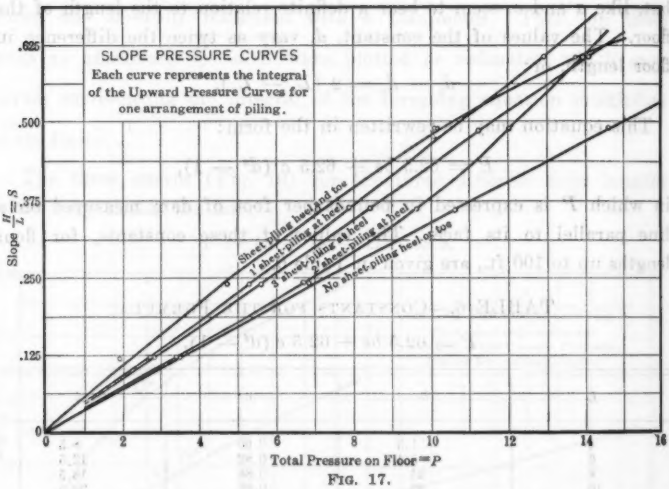


FIG. 17.

Taking up the curves (Fig. 17) for the sheet-piling at the heel, it is seen that these (unlike those for different floor lengths without sheet-piling, where P increased more rapidly than S) give results in which P increases less rapidly than S. The law governing this action and its association with the condition of no sheet-piling is not a simple relation. An approximate formula, which can be easily applied, is:

$$P' = P\,kp_1.$$

Now, if this equation is combined with the previous one for P, we have:

$$P' = [62.5\,bs + 62.5\,c\,(d^2 - 1)]\,kp_1.$$

The values of k are given in Table 7, from which it will be seen that the product, kp_1, changes but little with the change of slope for

values of p_1 greater than five; or, a 5-ft. depth of impervious cut-off wall is the practical limit in the reduction of the upward pressure on the floor of a dam.

TABLE 7.—VALUES OF THE CONSTANT, k, FOR THE FORMULA: $P' = [62.5\ bs + 62.5\ c\ (d^2 - 1)]\ k\ p_1$.

p_1	Slope 60 : 100	Slope 48 : 100	Slope 36 : 100	Slope 24 : 100	Slope 12 : 100
1	0.65	0.69	0.73	0.77	0.81
2	0.36	0.40	0.44	0.46	0.48
3	0.24	0.25	0.26	0.27	0.28
4	0.18	0.18	0.19	0.19	0.20
5	0.14	0.14	0.14	0.14	0.15

Taking up the condition of sheet-piling at both heel and toe, it is seen that P increases more rapidly than S. For low values of S, P is less than for the condition of no piling, but increases more rapidly as S increases, and crosses the first curve.

SEEPAGE.

With each pressure observation the temperature and discharge per minute were recorded. These discharges when corrected for temperature afford a partial means of comparing the effects on the quantity of seepage for different floor lengths and sheet-piling conditions, but are of no use in determining the law that governs the underflow. This law depends, not only on the length of the overlying impervious layer, the enforced length of flow, and the head of water, but on the porosity, the effective size of the sand grains, the uniformity coefficient, and the viscosity of the water. These factors are all very complicated, and at present are so little understood that an accurate formula is impossible.

The writer attempted to determine this law, in connection with his work, but was unsuccessful. The method used imitated that used by Schlichter for determining the velocity of underground water. Through the center of each of sixteen tubes scattered throughout the box a rubber insulated wire was passed, from the end of which the insulation for about 2 in. was cleaned. This bared wire, with the end of the pipe, formed two electrodes. The circuit was completed by connecting it with the lighting system, in one lead of which an ammeter was introduced.

The operation, as arranged and carried out, consisted of making readings of all tubes at 5-min. intervals. After two or three readings had been made, ordinary rock salt was introduced into the water above the dam. The 5-min. observations were continued until all traces of the salt had disappeared. These observations were then plotted, with times as abscissas and currents as ordinates, the peaks of the curves being taken to indicate the time required for the salt solution to reach the tube. The time required for the flow to pass from Tube a to Tube b would be $t_b - t_a$, and the velocity of flow over the distance, d, would be $\frac{t_b - t_a}{d}$.

In many of the readings this gave very high velocities under the toe of the dam, where the flow lines were well distributed; in fact, it frequently gave negative results, showing that the solution had passed by one tube and reached the second before influencing the reading of the first tube. After several attempts, these electrical observations, because of the unsatisfactory results and the undesirability of interfering with the pressure experiments, were discontinued.

However, the trials were sufficient to point out a more satisfactory method of attacking the problem. Each tube carrying an electrode should be arranged so that the salt solution could be charged at that point. Then, by charging one tube at a time and making readings as before, the time required for the flow to pass 1, 2, or 3 ft. could be determined. In arranging the tubes, care should be taken to make it unnecessary to trace any one charge for a greater distance than 3 ft.

Conclusions.

The conclusions resulting from this investigation may be stated as follows:

1.—The loss of pressure head of water in passing from above to below a dam may be divided into three stages:

(*a*) A relatively large loss of head, occurring where the water enters the sand;

(*b*) A quite uniform and comparatively small loss of head per foot of length through the body of the sand;

(*c*) A larger loss of head, occurring where the water leaves the sand.

2.—The first stage or entrance loss is increased as the length of floor is decreased.

3.—The rate of flow decreases very rapidly with depth below the floor of the structure, being more concentrated for shorter floors.

4.—The flow lines, once established in a porous medium, tend to continue horizontally as such, and come to the surface only after traveling a relatively great distance.

5.—The pressure decreases with the depth in front of the dam, and below the structure it reverses itself, increasing with the depth.

6.—The porosity, effective size, and uniformity coefficient have little influence on the upward pressure exerted against the floor of a dam.

7.—The pressure curve for the condition of no piling closely resembles the probability curve, having the general form

$$y = \frac{h}{\sqrt{\pi}} e^{-h^2 x^2}.$$

8.—Piling at the heel of a dam reduces the pressure on the floor, and piling at the toe increases it.

9.—With sheet-piling at the heel, the pressure increases less rapidly than the hydraulic slope. With no piling, and with piling at both heel and toe, the pressure increases more rapidly than the hydraulic slope.

10.—With sheet-piling at the heel, the flow of greatest density approximates the shortest distance from the end of the piling to the toe of the structure.

11.—The center of pressure, with no piling and with piling of any length at the heel, is $\frac{L}{3}$, measured from the heel of the dam. With piling at both heel and toe, the center of pressure is $\frac{5\,L}{11}$ from the heel of the dam.

12.—For the condition of no piling, the total pressure (in pounds) exerted on 1 lin. ft. of floor is:

$$P = 62.5\ b\ s + 62.5\ c\ (d^2 - 1).$$

13.—For the condition of impervious piling at the heel, the total pressure, in pounds, on 1 lin. ft. of floor is:

$$P' = [62.5\ b\ s + 62.5\ c\ (d^2 - 1)]\ k\ p.$$

14.—Piling at the heel of a dam, to be effective, must be tight, very small leakage destroying its action.

15.—Piling at the toe of a dam should be loose, in order to prevent increasing the upward pressure on the floor of the structure.

16.—An impervious cut-off wall of comparatively shallow depth, say 5 ft., at the heel of the structure will greatly increase its stability by reducing the upward pressure on the floor.

17.—Variation in the weight or downward pressure against the floor of a dam has little effect in changing the intensity of the upward hydrostatic pressure.

18.—The law of seepage or underground flow cannot be ascertained by treating an area as a unit. It can be best studied by breaking the area up into very small elements.

19.—The law, $P = a\,10^{-kx^n}$, when corrected for gravity, will probably apply equally to pressures in a vertical or horizontal plane.

ACKNOWLEDGMENTS.

The writer wishes to express his thanks to H. W. King, M. Am. Soc. C. E., for his valuable suggestions and assistance, to Professor Theodore R. Running for his timely assistance in solving the equations, and to Mr. Raymond A. Hill for aiding in the operations of the investigation.

DISCUSSION

Mr. Muckleston.

H. B. MUCKLESTON,* M. AM. SOC. C. E. (by letter).—This very interesting paper is most valuable, especially the curves presented as the result of actual experiment, as compared with those derived from purely theoretical investigation. The problem is susceptible of rigid mathematical analysis, if certain assumptions are made. These are: First, that the permeable base extends to an infinite distance in each direction in a horizontal plane; second, that the depth of the stratum is infinite; third, that it is uniform in permeability throughout and in any direction; and fourth, that the flow is capillary. Under these conditions, if a cut-off wall is considered as existing under the center of the impermeable base, of any depth, a, and the base width is $2b$, then the contours of equal pressure are the series of confocal hyperbolas:

$$\frac{x^2}{\cos.\ \beta} - \frac{y^2}{\sin.\ \beta} = c^2$$

where

$$c^2 = b^2 - a^2$$

and the lines of flow normal to them are the series of confocal ellipses:

$$\frac{x^2}{\cos.h\ \alpha} + \frac{x}{\sin.h\ \alpha} = c^2.$$

It is not necessary for the cut-off wall to be at the center, but if it is placed anywhere else, the equivalent depth at the center must be calculated from the equation.

It will be interesting to consider why the experimental results differ so widely from the theoretical. In the first place, the areas exposed to pressure between the ends of the box and the toe and heel of the experimental dam are very small in proportion to the base width of the dam; second, the depth of the permeable stratum is small compared to the base width; and third, the tail-water surface was somewhat above the sand level, whereas the theoretical hypothesis requires it to be at that surface.

The first and third conditions are probably responsible for the observed entry and exit heads, and the second is the cause of the peculiar shape of the pressure curves. The latter is the more noticeable when the experiments with the cut-off wall are considered, especially in those with the 3-ft. wall, where the throttling effect of the narrow passage is clearly seen. This effect is evident in all the plotted experiments, and it is very probable that had the observed pressure points been more closely spaced, island pressure contours would have been detected, for example, in Fig. 2, Exp. No. 38, Obs. No. 4, April 19th, 1914, between Points 19 and 29.

* Calgary, Alberta, Canada.

Mr. Muckleston.

Another effect of the first condition is noticed most strongly in Fig. 7, Exp. No. 20, Obs. No. 3, March 24th, 1914, where the pressure contours up stream from the cut-off are seen to be nearly parallel and approximately horizontal. The sharp downward curvature of the 2.50 contour is also noticeable, and is probably due to its very close proximity to the foot of the wall and the constricted opening under it.

Although the assumed conditions of the theoretical calculations are probably very far from those obtaining in actual practice, it is believed that the conditions of the experimental determinations are about equally far in the opposite direction, the truth lying somewhere between them.

Mr. Travell.

WARREN B. TRAVELL,* ASSOC. M. AM. SOC. C. E. (by letter).—This interesting paper bears directly on a question which is not only very important, but concerning which prominent engineers of the present day have radically different opinions—the question of uplift on the base of a concrete or masonry dam of the gravity type.

There is no doubt that, in the past, many dams have been built without any allowance having been made in their calculations for uplift, which is equivalent to taking the upward pressure on the base of the dam as zero, although a few engineers are insisting that the uplift should be taken at 100%, or full hydrostatic pressure.

These experiments constitute a valuable addition to our scanty knowledge of the subject, but it must be admitted that they were made on a rather small scale, and it is to be hoped that they will be added to by others, so that before long sufficient data will be furnished whereby engineers may come to an agreement in this matter.

It is noted that, in his experiments, the author used three different lengths of bases, namely, 4 ft. 3 in., 6 ft. 3 in., and 8 ft. 3 in., and on these short lengths as a foundation, he proceeds to draw conclusions applicable to all lengths of bases up to 100 ft.

The values of the constant, b, in the formula deduced for pressure, are determined to be 1.49, 6.13, and 24.78, for these three base lengths, and from these data, values of b up to 2 587 are derived. This appears to be equivalent to attempting to draw a curve having given only three points in that curve. The length of the curve thus given is the difference between 24.78 and 1.49, or 23.29 units, and this curve is then extended beyond the farthest given point to a length of 2 562 units. In other words, the curve is extended to a distance 110 times the determined length.

In the first place, three points are not sufficient to determine a curve of this kind, even for the distance between these points, and the extension of the curve thus found, especially when carried out to such a length, is interesting as the record of a guess, but is hardly entitled

* New York City.

Mr. Travell.

to a place among scientific data. Some of the greatest engineering disasters have resulted from the drawing of extended conclusions based on limited tests, and too great emphasis cannot be laid on the warning against such practice.

When frequent repetitions of an experiment are made under the same conditions, the uniformity of the results determines largely the degree of confidence to which such experiment is entitled. Examination of the tabulated pressure heads, given in Table 2 (Plate XV), apparently shows considerable variation for experiments conducted under similar conditions. For instance, in Experiment No. 29, Observation No. 1 was made with water at a height of 2.46 ft. above the base, and, in Observation No. 2, this was slightly less, or 2.44 ft. Yet the pressure heads given in the latter observation are as much as 36% greater than those given in Observation No. 1.

Regarding the curves for upward pressures, without sheet-piling, as shown in Figs. 9, 10, and 11, it is not clear to the writer why there should be about 80% of the 5-ft. head at the heel of the 4.25-ft. and 8.25-ft. dams and only 58% at the heel of the 6.25-ft. dam. It would seem that this figure should be practically constant for any given head, irrespective of the length of base.

Also, in Figs. 12, 13, and 14, showing pressures with different lengths of sheet-piling at the heel, it appears strange that, comparing the curves for the 5-ft. head, the pressure should increase from 49% of the head for 1-ft. sheet-piling to 66% for 2-ft. sheet-piling and to 76% for 3-ft. sheet-piling. As the sheet-piling was made longer, a decrease in pressures would naturally be expected, rather than an increase as shown in these curves.

It would be of great value to engineers if experiments similar to these were made in connection with the actual building of some masonry or concrete dams of the gravity type. Naturally, there would be a wide variation in the results of such experiments, depending on the porosity of the underlying foundation, the presence of seams in a rock foundation, the density of the concrete, and many other causes, but data thus obtained and given to the Profession would eventually settle the debated question of uplift.

Mr. Elliott.

MALCOLM ELLIOTT,* ASSOC. M. AM. SOC. C. E. (by letter).—For several reasons, the equations developed from this series of experiments do not seem to be suitable for general use in determining the upward pressure on the base of a dam built on sand foundation.

There is no known law of physics or hydraulics which would indicate that the pressures under a dam will be expressed by an equation in the form of $P = 62.5\ [bs + c\ (d^s - 1)]$. The author has measured the pressure on the model dam, platted the points, and drawn through them a curve which he finds is approximately the graph of the equation

* Louisville, Ky.

Mr. Elliott.

stated. So far as the writer knows, there is no reason that the pressures under a dam should be expressed by the curve of the law of error, and the fact that, within certain limits, the pressures are approximated by this curve should not be taken to indicate that the equation is a general expression for the pressure for all heads and floor lengths.

The utmost that can be said for the author's equation is that it gives fairly well the pressures which will occur in a model of the same dimensions and founded on the identical material used by him.

To demonstrate that the equation is not generally applicable, it might be well to cite some experiments carried on recently under the direction of J. P. Jervey, M. Am. Soc. C. E., Major, Corps of Engineers, U. S. Army, and described in an article* by Capt. W. A. Mitchell, Corps of Engineers, U. S. Army. The model used in these experiments was very similar to that used by the author. The floor length was 9 ft., made up of nine concrete slabs, 1 ft. wide, the joints of each being caulked. The heads were from 0 to 5 ft. Instead of having a flat bottom, the box containing the sand and the model dam was recessed so as to allow about the same space under the piling as under the slab. The piling was at the heel, and was 2 ft. long. In these experiments, three different materials were used, the following description of which is quoted from Capt. Mitchell's article:

"'Sand' means washed river sand screened through a ¼-inch screen.

"'Washed gravel' means ordinary river gravel washed and screened through a 2-inch, and retained on a ¼-inch, screen.

"'Gravel aggregate' means ordinary river-bottom mixture of gravel, sand, and mud at the site of Dam No. 14 [Ohio River], without any washing or screening whatever.

"'Alluvial soil' means ordinary sedimentary deposit on the banks of the river, as was obtained from the field near the apparatus."

The pressures transmitted through these materials are given in Table 8, which was compiled by the writer from Capt. Mitchell's data, the pressures having been converted from inches in the piezometer tubes to square feet of pressure curve, so as to afford a comparison with the pressures as computed from the author's equation. To obtain the actual pressure, in pounds, the quantities in Table 8 should be multiplied by 62.5.

The most striking point brought out by this comparison is the difference in pressures for different materials. This evidence does not seem to substantiate the author's Conclusion 6, namely:

"The porosity, effective size, and uniformity coefficient have little influence on the upward pressure exerted against the floor of a dam."

*Professional Memoirs, No. 31 (January–February, 1915), Corps of Engineers, U. S. Army, Washington Barracks, D. C.

TABLE 8.—COMPARISON OF PRESSURES TRANSMITTED THROUGH VARIOUS MATERIALS. Mr. Elliott.

Head, in feet.	Slope $\frac{H}{L}$	Observed Pressures, U. S. Engr. Experiments.				Pressures computed from author's equations.
		Sand.	Gravel aggregate.	Washed gravel.	Alluvial soil.	
0.125	0.0139	....	....	0.84	....	0.51
0.208	0.0231	....	....	1.32	....	0.75
1.333	0.1481	3.92	....	....	....	5.43
1.5	0.1667	....	0.56	....	....	6.00
2.5	0.2778	6.90	....	....	....	9.73
3.0	0.3333	....	1.00	....	0	11.5
3.5	0.3889	7.88	....	....	....	13.3
4.0	0.4444	8.21	....	....	0	14.2
5.0	0.5556	....	1.77	....	0	16.6
5.33	0.5922	....	....	....	0	17.8

Notwithstanding the difference in pressures shown by these two independent sets of experiments, due apparently to differences in porosity, etc., the writer, for practical purposes, would be inclined to accept the author's conclusion, because the influence of porosity, etc., if any, is not well understood, and cannot be determined without large-scale experiments at every dam site, and even then it would not be certain that the model used would duplicate the actual dam. For instance, the space under the model dam and the outlet through which the water flows are necessarily restricted in area, though the sand under a dam may be very deep and the outlet unlimited in extent. Who can say that these differences between "laboratory" and actual conditions do not materially affect the pressures?

It is probably because of the uncertainty as to the influence of porosity, etc., that, as a rule, the upward pressures are computed as if these factors had no influence. The fact, however, that these two sets of experiments give different pressures would indicate that an equation derived from experiments on a small-scale model and fashioned after no recognized physical law, cannot be considered generally accurate.

It is not necessary to call in outside evidence, however, to discredit the author's equations. They convict themselves. Extend the curve, in Fig. 16, for the 4.25-ft. floor, and it will be seen that for $\frac{H}{L} = 1.75$, the area of the pressure curve will be about 33 sq. ft., giving a pressure of about 2 060 lb. It will be granted without question that in no case can the upward pressure exceed the full head acting over the entire base. For $\frac{H}{L} = 1.75$, this would amount to about 1 950 lb. Thus, if the equation is true, we would have the surprising condition of an upward pressure greater than the head, and the water in the

Mr. Elliott.

tubes would stand higher than the water surface above the dam. For values of $\frac{H}{L}$ greater than those cited, the computed pressures increase very rapidly, and greatly exceed the total head. With the 6.25-ft. and 8.25-ft. floors, equally absurd results are obtained by extending the curves to $\frac{H}{L} = 1.7$ and $\frac{H}{L} = 1.6$, respectively.

It is the writer's opinion that the safest and most convenient assumption for the upward pressure and the location of its centroid is that there is a uniform loss of head from full head to zero throughout the distance the water travels in percolating under the dam. The distance traveled in the case of a dam with sheet-piling at the heel is approximately twice the length of the piling plus the width of the dam, and this sum divided by the head will give the hydraulic gradient or slope of the "piezometric line".*

Although this method may not be strictly correct, on account of making no allowance for differences in porosity, uniformity coefficient, and effective size, it will, so far as known, give results which are on the safe side. At the same time, the effects of porosity, etc., if any, are so vaguely understood that it is practically impossible for the designer to determine what they are.

Usually this method will give pressures greater than those recorded in the experiments referred to herein, but, in view of the uncertainties previously mentioned, it is not believed that the margin of safety will be unreasonably large, and the experiments are not conclusive enough to justify a departure from a method which, in the past, so far as known, has proved safe.

The action of water under dams is not limited to the upward pressure that it will exert on the base. A dam may be safe against overturning and sliding and yet fail on account of the erosion of the material under it, due to the flow of water through the porous material. This is perhaps the most uncertain of all conditions that must be considered in the design of a dam on a granular foundation. An investigation of the action of water under dams, to be thorough, should certainly deal, to some extent, with erosion.

Mr. Hays.

JAMES B. HAYS,† JUN. AM. SOC. C. E. (by letter).—During the past 15 months the writer has been studying along just such lines as those followed by Mr. Colman in his paper, and it gives him great pleasure to examine the results obtained by others, which, in a large measure, corroborate those obtained by himself.

A few months ago he began to write a paper on the subject for presentation to the Society, but press of other work and the desire to make further investigations have prevented its completion.

* The method is explained very clearly by W. G. Bligh, M. Am. Soc. C. E., in *Engineering News*, December 29th, 1910, p. 708.

† Boise, Idaho.

Mr. Hays.

The object of the writer's experiments was to determine the dimensions of an earthen dam to be constructed on a river bed of deep gravel. They were conducted in a large wooden tank, with an actual model of the proposed dam, constructed to scale, built on the gravel. Sheet-piling of various depths was interposed at different locations. Glass tubes communicating with the interior of the gravel mass showed graphically the pressures at the different points. The dam was placed about at the middle of the tank, and water was introduced on the up-stream side. Readings were taken showing the pressures at the different tubes for each of several different heads of water on the up-stream side. Seepage was also computed for each of the different heads.

In view of the writer's experience, he agrees with the author on the following points: (*a*), That there is a large loss of head where the water enters the gravel; and (*b*) that a practically uniform and relatively small loss of head per foot occurs horizontally through the main body of the gravel where its cross-section is uniform.

Regarding the loss of head where the water leaves the sand, or gravel, the writer's experiments and those of the author did not give the same results. However, this may be explained in either one, or both, of two ways.

The author's model was constructed so that the water entered the sand in a space 4 ft. wide, traveled under the dam in a space 5 ft. wide, and then, in the case of the 8-ft. floor, issued through a space 2 ft. 11 in. wide. It will be noted that the point of entry and the point of egress are constricted sections, relative to the space under the dam, which would naturally cause the water to be retarded, showing thereby a greater pressure head. The writer avoided these constrictions, and believes that his model more nearly represents the case in actual practice.

Then, again, where the shorter floor was used, the entire pressure head was not consumed. In some of the experiments using the longer sheet-piling, the base of the writer's model was longer in proportion to the head of water supplied, and without the constrictions mentioned, all the initial head was found to be consumed in friction, etc., before it issued from the down-stream side. Given a definite head of water, acting on a body of sand or gravel with known characteristics, a definite velocity will be produced. This, in turn, means a certain loss of head per linear unit and, by extending the length of the body of sand, or gravel, a point is reached where all the initial head is consumed. This is shown by the intersection of the hydraulic grade line with the body of the sand.*

* The writer would refer, in this respect, to Buckley's "Irrigation Pocketbook", p. 362, *et seq.*, and, also, to Professional Memoirs, U. S. Engineer Corps, Vol. VII, p. 32, *et seq.*, especially to p. 44.

Mr. Hays.

It is the writer's belief that the head lost by the water entering the sand might not have been so great had the width of the section been 5 ft. or more, instead of 4 ft. However, his own experiments show a large loss of head at the entrance of the water into the sand, or gravel. This, in all probability, represents the entry head and the velocity head.

The author's Conclusion 2 might be explained by the fact that with a shorter floor he has produced a steeper hydraulic gradient, hence a larger head is lost, creating the increased velocity necessary.

In regard to Conclusion 5, does the author mean that the pressure increases with the increase of depth, or the decrease of depth?

In general, aside from the points mentioned, the writer's experiments agree with those of the author, and though they do not reach the same conclusions on some points, this is explained, in all likelihood, by the difference in construction of the models.

Mr. Creager.

W. P. CREAGER,* M. AM. SOC. C. E.—The author deserves a great deal of credit for his work in connection with these experiments. The curves showing equal-pressure contours under his model dams will be of great interest to engineers and of material assistance in connection with further studies on the subject.

The speaker feels the necessity, however, of calling attention to the danger of applying information gathered from experiments on a small scale to the design of structures of much greater magnitude. In order to indicate the grave discrepancies which may be expected to result from the application, to large dams, of the author's formula for uplift, the speaker has computed from this formula the uplift on a dam with a 20-ft. base when subjected to a 30-ft. head of water. The author's formula follows:

$$P = 62.5\ bs + 62.5\ c\ (d^s - 1).$$

For this case, $H = 30$; $L = 20$; $S = 1.5$; $b = 278$; $c = 0.94$; and $d = 40.5$, from Table 6.

Therefore,

$$P = 62.5 \times 278 \times 1.5 + 62.5 \times 0.94\ (40.5^{1.5} - 1) = 41\,200.$$

The average uplift on the base is

$$p = \frac{P}{L} = \frac{41\ 200}{20} = 2\ 060 \text{ lb. per sq. ft.}$$

The average uplift head is $h = \frac{2\ 060}{62.5} = 33$ ft.

$$\frac{h}{H} = \frac{33}{30} = 1.10.$$

* New York City.

Mr. Creager.

From which it is seen that, for this example, the author's formula indicates a 10% greater average uplift head on the base than the head on the dam. This, of course, is an impossibility, as the greatest uplift head which could possibly obtain would be exactly equal to the head on the dam, and the average uplift head could equal the head on the dam only if there was an impervious barrier at the toe of the dam.

The speaker has calculated from the author's formula and plotted on the diagram, Fig. 18, the average uplift head, in terms of head

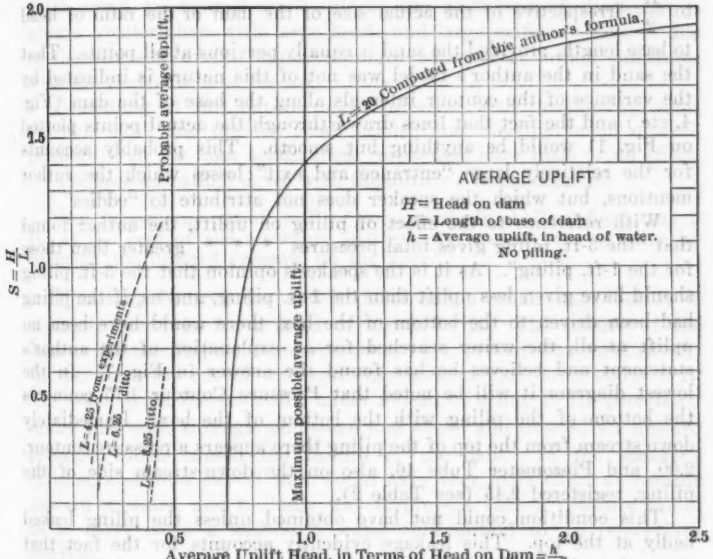


FIG. 18.

on the dam for a 20-ft. base and various values of S. He has also plotted two vertical lines representing the maximum "possible" uplift head and the "probable" uplift head for foundations of this type. The value, 0.5, for the speaker's probable uplift, was obtained by assuming an uplift head equal to H at the up-stream end of the base and diminishing uniformly to zero at the toe.

It is seen that the curve computed from the author's formula gives values of uplift at all points greater than the speaker's probable value, and for S larger than 1.4, it gives greater values than it is actually possible to obtain. Values of S greater than about 1.4 are not practical for dams with considerable uplift (unless built with a great super-elevation above water surface) and this does not alter the fact that

Mr. Creager.

the author's formula for such conditions is incorrect. There is, then, some question as to the accuracy of the formula for lower values of S.

The direct results from the author's experiments are plotted in dotted lines on the diagram. They all give values of average uplift less than the speaker's probable value, which is a condition quite likely to occur.

The speaker believes that the average uplift head on the base of an ordinary dam on a sand foundation without a cut-off is very close to $\frac{H}{2}$, irrespective of the actual size of the dam or the ratio of head to base length, provided the sand is equally pervious at all points. That the sand in the author's model was not of this nature is indicated by the variance of the contour intervals along the base of the dam (Fig. 4, etc.) and the fact that lines drawn through the actual points plotted on Fig. 11 would be anything but smooth. This probably accounts for the relatively large "entrance and exit" losses which the author mentions, but which the speaker does not attribute to "eddies".

With reference to the effect of piling on uplift, the author found that "the 3-ft. piling gives total pressures * * * greater than those for the 1-ft. piling". As it is the speaker's opinion that the 3-ft. piling should have given less uplift than the 1-ft. piling, and as, if the piling had been driven to the bottom of the box, there would have been no uplift at all, the writer searched for an explanation of the author's statement and believes he has found the answer in Fig. 7. In the lowest diagram it will be noted that Pressure Contour 1.75 connects the bottom of the piling with the bottom of the box. Immediately down stream from the top of the piling there appears a pressure contour, 2.00, and Piezometer Tube 46, also on the down-stream side of the piling, registered 2.45 (see Table 2).

This condition could not have obtained unless the piling leaked badly at the top. This leakage evidently accounts for the fact that the author observed greater uplift for 3-ft. piling than for 1-ft. piling, the latter being tight, as shown by Fig. 5.

Mr. Wegmann.

EDWARD WEGMANN,* M. AM. SOC. C. E.—The speaker must compliment the author and the University of Michigan on the scientific manner in which the experiments on the action of water under a model dam have been made and the results worked up. These experiments, however, were on a very small scale, and very different results may occur in the case of a dam having to sustain a pressure due to a large head. In these experiments, the greatest head was only about 5 ft. The base of the model dam was only 8 ft. 3 in., and the flow of the tail-water was affected by the end of the test box, which acted like a sheet-piling. The speaker does not consider it safe to apply to the case of a high dam some of the conclusions of the paper.

* New York City.

Mr. Wegmann.

The author found that the porosity, effective size, and uniformity coefficients had little influence on the upward pressure exerted against the dam. The speaker thinks that these factors would have a very material effect on such pressure, except for a very low head, in which case viscosity of the water, capillary action, etc., might prevent the water from exerting much upward pressure.

As an instance of the difference between the effect of high and low heads, attention is called to experiments with the flow through orifices. With small heads and small orifices the coefficient of discharge is always greater than with large heads and large orifices. This is due to the viscosity of the water, capillary action, etc., which interferes with the contraction of the jet.

Something of this nature might interfere with the flow of water through sand, and prevent small heads from exerting much upward pressure; but, for high heads, these resistances have little or no effect.

The speaker cannot agree with the author's Conclusion 16:

"An impervious cut-off wall of comparatively shallow depth, say 5 ft., at the heel of the structure will greatly increase its stability by reducing the upward pressure on the floor."

This may be true for a 5-ft. head and a small base, but it is certainly not true for a head of about 50 ft. The failure of the dam at Austin, Pa., on September 30th, 1911, proved that conclusively. In this case, the cut-off wall had only been carried down about 6 ft. The water from the reservoir, under a head of about 45 ft., percolated under this wall, through sandstone and shale, and exerted an upward pressure which caused the failure of the dam. In the speaker's opinion, a shallow cut-off wall would have very little effect as regards upward pressure, and the only way to prevent such a pressure would be to carry down a well-built, water-tight cut-off to an impervious stratum.

Recently the speaker has had some interesting experiences relative to the manner in which pressure may be transmitted under the base of a dam. In the case he has in mind, an earthen dam, about 25 ft. high, had to be constructed for about 250 ft. of its length on soft, marshy ground, underlain at the depth of 27 ft. by sand. The dam was 10 ft. wide on top. The up-stream side was sloped 2 to 1, but the slope was broken 14 ft. above the base by a berm 5 ft. wide. On the down-stream side a similar berm, 6½ ft. wide, was placed about 12 ft. above the base. Below this berm, the down-stream slope was made 2 to 1, but above it, the slope was only 1½ to 1. A core-wall, founded on a pile foundation, was constructed in the center of the earthen dam, and, in order to prevent the water from leaking under the dam, tight sheet-piling was driven on the up-stream side of the pile foundation.

Mr. Wegmann.

The dam would have stood successfully, if it had been founded on hard ground, but, for the marsh that had to be crossed, the plan adopted had the great defect of placing a considerably greater pressure on one side of the core-wall than on the other. In the former case the pressure was caused by the water and the saturated earth on the up-stream side of the core-wall. In the latter case the pressure was due to the dry earth on the down-stream side of the core-wall. The difference in pressure was transmitted through the marshy ground almost like the hydrostatic pressure of water, and forced the foundation platform of the core-wall and the sheet-piling down stream. The whole core-wall followed the same movement, and, when the speaker was consulted about what remedies should be adopted, the top of the core-wall had moved more than 3 ft. down stream.

A simple calculation showed the inequality of the forces acting on the two sides of the core-wall, and the speaker recommended an increase of 15 ft. in the top width of the dam and making the down-stream slope 3 to 1. These measures were carried out, and no further trouble has been experienced.

In the speaker's opinion, the proper way to have crossed the marsh would have been by constructing a homogeneous earth dike, having a greater slope on the down-stream than on the up-stream side, so as to offset the weight of water on the up-stream slope. The dam was bound to settle, and the only remedy for this was to continue filling until the soft material under it had been sufficiently compressed to bear the weight of the embankment.

If sheet-piling had been required to prevent leakage under the dam, it should have been driven, either at the toe of the up-stream slope, or on the center line of the dam, after the material under it had reached a state of equilibrium.

The speaker had a similar experience on the New York, West Shore and Buffalo Railroad, which showed how pressure was transmitted through soft ground. In this case two abutments, each 45 ft. high, had to be built for a 60-ft. plate-girder bridge which spanned Cedar Pond Creek, near West Haverstraw, N. Y. The ground at the site of the bridge was very similar to that on which the dam just mentioned had to be constructed. A long embankment, 45 ft. high, carried the railroad from the south side of the valley to the south embankment. A foundation was prepared for this embankment by corduroying the ground with two courses of round timber. On the north side of the bridge, only a short embankment was required. Both abutments were built on pile foundations.

When the toe of the south bank had about reached the south abutment, the weight of the embankment, although well distributed by the corduroy timbers, caused a pressure on the soft ground, which

forced the piling toward the creek. A similar movement occurred in the north abutment. The displacement of the two abutments, which reduced the original span by about 1½ ft., was due partly to scouring in the creek bed, which increased the depth of water from about 2 to 6 ft. Mr. Wegmann.

The moving of the abutments was stopped temporarily by bracing one against the other by 60-ft. piles, bolted together in pairs, tip to butt. A permanent bracing of 12 by 12-in. timbers, looking like an inverted roof, was then placed between the abutments, the peak resting on a crib sunk half way between them. Further scouring was prevented by driving sheet-piling above and below the abutments. The space between the two rows of sheet-piles was filled with stones. After the work had been completed, the temporary bracing was removed.

No further trouble was experienced with the abutments, and for many years they were held safely by the bracing described. The speaker understands that some changes or repairs have recently been made to the abutments, but he does not know their nature.

W. R. BALDWIN-WISEMAN,* ESQ. (by letter).—On page 430 the author has summarized the work of a few of the more important investigators of this subject; but, on a subject about which there are such extreme variations of opinion, it would have been better to have given a more detailed list. Schlichter† supplied a much more complete one, and the writer has given fuller details in the three following papers: "The Flow of Underground Waters,"‡ "Statistical and Experimental Data on Filtration,"‖ and "The Influence of Pressure and Porosity on the Motion of Subsurface Water."¶ Incidentally, another aspect of the subject is dealt with in "The Influence of the Thickness of the Pipe Wall on the Rate of Discharge of Water from Minute Orifices Piercing the Pipe."** Mr. Baldwin-Wiseman.

A very careful investigation of the subject has also been undertaken, and full references to other work are given, by J. Versluys in "Le Principe du Mouvement des Eaux Souterraines."††

The subject has been dealt with by the writer in "Statistical and Experimental Data on Filtration" (quoted previously), more particularly on pages 20 to 40, 55 to 61, and 127 to 130.

Data in the writer's possession, but as yet unpublished, show that the rate of interstitial flow and of loss of pressure are largely dependent on the material of the sand grains, on their angles—that is, whether

* Southampton, England.
† U. S. Geol. Survey, 19th Annual Report.
‡ *Minutes of Proceedings*, Inst. C. E., Vol. CXLV, 1905–06, pp. 309–352.
‖ *Minutes of Proceedings*, Inst. C. E., Vol. CLXXXI, 1909–10, pp. 15–157.
¶ *Quarterly Journal*, Geol. Soc., Vol. LXIII, 1907, pp. 80–105.
** *Transactions*, Assoc. Water Eng., Vol. XII, 1907, pp. 373–380.
†† Published in Amsterdam in 1912.

Mr. Baldwin-Wiseman.

they were formed by subaerial or submarine denudation—and on the relative homogeneity of the sand mass; and the results of experiments conform more closely to an arbitrary law when the sand grains composing a mass are of fairly uniform size than when the constituents of the mass are heterogeneous, even though the mean diameter of the grains of the mass, when determined by a size analysis, may be almost identical with that of the uniform or graded sand. This point is well illustrated in Fig. 2 of the paper "Statistical and Experimental Data on Filtration". In that paper the writer has endeavored to represent these factors of roughness, angularity, etc., by a term which (for lack of a better one) has been called the thickness of the ideal film, not because such a film exists, but because it is a facile conception, and, as will be seen by consulting the table on page 33 of that paper, sands with practically the same porosity have ideal films, varying in the ratio 1:0, 1:7 and 0:4. So much for the velocity of interstitial flow. From unpublished investigations the writer has found that the greatest loss of pressure occurs in interstitial passages which have extreme and rapid change in cross-section.

The writer concurs entirely with Conclusion 1, as it is in agreement with conclusions succinctly stated on page 319 of his paper "The Flow of Underground Water",* and are well illustrated in the diagrams, *a* and *b*, on the same page; and they are fully confirmed by later experiments (not yet published) with graded and natural sands similar to those described in "Statistical and Experimental Data on Filtration".†

The writer also agrees with the substance of Conclusions 2, 3, and 4, as his own work has led him to similar conclusions, but he cannot agree with the deductions of Conclusion 6, for reasons already detailed herein. This, however, is doubtless due to the method of arriving at the effective size and uniformity coefficient, a subject on which the writer has already expressed an opinion and indicated better methods in his paper on filtration.

Mr. Oakes.

JOHN C. OAKES,‡ M. AM. SOC. C. E. (by letter).—This paper is of particular interest to those engaged in the design and construction of river improvement works.

In the writer's opinion, the model used in the experiments might have been improved by increasing its size, giving greater space beneath the sheeting, and more room for the escape of tail-water. The contraction beneath the sheeting, and the short distance between the ends of the platform and box, may have had considerable effect on the resulting pressures.

As might be expected, perhaps, the data in Tables 1 and 2 (Plates XIV and XV) show some inconsistencies which are more or less

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXV.

† *Minutes of Proceedings*, Inst. C. E., Vol. CLXXXI.

‡ Major, Corps of Engrs., U. S. A., Louisville, Ky.

puzzling. For instance, Tubes 2, 5, and 8 had their entrances at practically the same level, yet in some experiments the height of the water in Tube 8 was greater, and, again, in other experiments, much less than in Tubes 2 and 5. In Experiment No. 1, Observation No. 2, the values for Tubes 2 and 8 are, respectively, 0.50 and 0.93, yet, in Experiment No. 3, Observation No. 3, the values are 3.43 and 2.86. Mr. Oakes.

Figs. 9, 10, and 11 show pressure curves for different heads and different lengths of base. For a 5-ft. head, the pressure at the heel, with bases 4.25 and 8.25 ft. long, is greater than that for the base 6.25 ft. long. There is no logical reason for such a difference, and every reason why the pressures at the heel, under the conditions shown, should have been approximately the same; if any difference was to be expected, it would seem that the pressure should have been less for the shortest base, increasing to greatest for the longest base, on the theory that the slope of the pressure curve would be greatest when the path is shortest.

Figs. 12, 13, and 14 show pressure curves for the base 8.25 ft. long, with sheeting 1, 2, and 3 ft. long, under the heel. These curves show the smallest pressure at the heel when the shortest piling was used, and the greatest pressure when the longest piling was used. This is exactly the reverse of what should have been shown. The points plotted for Tubes 22, 26, 30, 33, and 36, for 2 and 3-ft. piling, are almost exactly in the same location for each case, and the pressures in Tube 16 alone show reduction for the 3-ft. piling, and there is an increase for Tube 46 for the same piling. By comparing Figs. 11 and 12, it will be noted that the pressures are very much reduced by using 1-ft. piling, particularly with the greater heads, and, logically, the longer piling should reduce the pressures still further.

These inconsistencies can probably be explained by the fact that sufficient time was not allowed to develop the pressures which should have been developed by the various heads recorded, and that there was some leakage through the sheeting. The author does not state how the head was held, but has mentioned that the time allowed for the head to adjust itself was not less than 2 hours. It seems probable that this time was not sufficient, otherwise it is difficult to account for the results. The author would have obtained much better and more consistent results if the head had been maintained at definite heights for 6 hours or more before readings were made.

However, the illustrations showing lines of equal pressures—Figs. 2 to 8, inclusive—are very interesting and instructive. They appear to confirm previous assumptions of the loss of head throughout the length of flow, and the effect of sheet-piling in increasing the length of the flow lines.

Mr. Oakes.

W. G. Bligh, M. Am. Soc. C. E., has stated* with reference to sheeting under dams, as follows:

"It has been ascertained experimentally that the value in increase of length of percolation due to a vertical obstruction is double that beneath a horizontal apron, for the reason that the percolating current travels down one side of the obstructing curtain wall and up the other."

The writer believes that this principle has been accepted, generally, for determining the length of travel for water under dams, when sheet-piles have been used. From the illustrations previously mentioned, it seems that a more conservative and correct length of flow would be obtained by taking the length of the sheeting below the river bed, plus the distance from bottom of sheeting to toe of base.

The writer cannot agree with the author's conclusions that the porosity, effective size, and uniformity coefficient have little influence on the upward pressure.

Other writers have shown the error in the author's equations for the pressure curve and for total pressures under the dam.

The most important of the author's conclusions, and the ones to be borne in mind by designing engineers, are that sheeting at the heel of the dam reduces the pressure under the floor, and that sheeting at the toe increases it; that sheeting at the heel of a dam, to be effective, must be tight (very small leakage destroying its action); and that sheeting at the toe should be loose in order to prevent increasing the upward pressure on the floor of the structure.

Although the academic discussion of pressures under a dam on sand foundation is of interest, the rules and conclusions deduced from experiments on models must not be allowed to affect too greatly engineering practice in the designing of actual structures. The use of the model in this case introduced several conditions which do not exist under actual structures, such as practical uniformity of sand, limitation of escape of seepage by bottom, sides, and end of box, and contraction under sheeting.

Practical rather than theoretical considerations are of the greater importance, because the conditions which the experimenter is able to obtain with a model can never be obtained in the field. For instance, it is noted that the author used a pressure on the sand under the platform of from 125 to 640 lb. per sq. ft. It may be that such pressures are developed under some dams on sand foundations, but such cases will be exceptional. Very seldom, if ever, will masonry dams or weirs be designed to rest directly on sand without supporting piles. Certainly, in river work, no one would think of building such structures on sand without using both round and sheet-piles, in which case there would be practically no pressure on the sand. If the bed of the founda-

* *Engineering News*, December 29th, 1910, p. 709.

tion is below the water level, and pumping is required, there will be a small space between the base of the masonry and the sand, caused by erosion by the flow of water from under one block of masonry while the next one is being placed. In other words, the structure is ordinarily supported wholly by the piles, with almost a certainty that there will be a space between the sand and the masonry. Under such circumstances, any upward pressure that develops under the dam will be uniform from the sheet-piling to the toe, and will depend on the tightness of the sheeting and the ease of escape below the dam, rather than on ideal laboratory conditions obtained with a model. Mr. Oakes.

Another practical consideration which must not be lost sight of is the difficulty, almost amounting to an impossibility, of obtaining a tight sheeting, unless excavation is made to the full depth and an impermeable wall is constructed. The writer is firmly convinced that not more than once in a hundred times will driven timber sheeting be tight. When interlocking steel piles are used, there is grave danger that the piles may be forced apart while being driven, destroying the effectiveness of the sheeting. Even when this does not occur, there will still be considerable leakage through the joints between the piles, particularly when first driven. Considering, therefore, that a tight sheeting is a very rare occurrence, and that most dams and weirs on sand will be supported on piles with a space between the masonry and the sand, full upward pressure must be assumed. Furthermore, if dams to be built on sand are properly designed for stability against sliding and overturning, and seepage is reduced sufficiently to prevent piping and blow-outs, by a proper combination of sheeting and width of base, the structures will be found, in most cases, to be capable of withstanding full upward pressure. Typical designs of certain structures, however, such as reinforced concrete dams, and the masonry bases of movable dams, will not provide sufficient weight to withstand such pressure, and the weight must be increased, or some other means, such as hold-down piles, must be used to insure stability.

Among the more important structures being built at the present time on sand foundations are the locks and dams in the lower Ohio River. These works are being built to provide a navigable depth of 9 ft. throughout the length of the Ohio. Movable dams (Chanoine) are being constructed with normal lifts of from 7 to 9 ft., and maximum lifts as great as 16 to 17 ft. The greater lift at a dam occurs when the lower pool has been lost, due to the maneuvering of the next lower dam, when there will be low water below and an upper pool above the dam under consideration. The locks are 600 by 110 ft., inside dimensions (usable). All the locks and dams below Louisville, twelve in number, must be built on sand foundations, and the means of preventing failure of these works, due to upward pressure, piping

Mr. Oakes.

and blow-outs, and undermining by erosion below the dams, have been of the greatest interest and anxiety to those engineers who are responsible for the design and the construction of these works, which are exceptional in size of locks, lengths of dams, and heads to be withstood by river structures on sand foundations.

During the last three years the writer has had charge of the construction of the Ohio River Locks and Dams Nos. 43 and 48, both being built on sand foundations. The typical plans provide for a timber sheeting 25 ft. deep, of Wakefield piles, to be driven under the toe of the land- and guide-walls, under both toe and heel of the river wall, and under both up-stream and down-stream edges of the sills. The plans originally called for the same kind of piling under the heel of the dam. All structures are supported on round and sheet-piles.

After the difficulties of driving timber piles to form a tight sheeting had been thoroughly proved, and full consideration had been given to the insecurity of structures on sand foundations, a line of interlocking steel sheet-piling, 40 ft. long, was substituted for the Wakefield piling under the dams, and a heavy lock floor, 4 ft. thick, held down by round piles, replaced a light floor, 18 in. thick, without piles. The writer has watched the construction of these two works for three years, has noted the careful efforts to produce a tight sheeting with the timber piles, and has concluded that it is almost impossible to do so. This confirms his previous experience and that of several other engineers with whom he has discussed this matter. Certainly, in the design of structures on sand foundations, the assumption cannot safely be made that a tight sheeting can be provided by driving timber sheet-piles. Better results can be obtained with steel piles, but one never knows when the sheeting is disrupted in driving, and there is always more or less leakage, even without disruption.

In the same manner the writer has watched the placing of concrete, and in no case has he found, on the works under discussion, that the concrete rested on the sand after sufficient time had been given it to set. The bed of the foundation of the various parts of these structures is about 10 ft. below low water. Construction has been carried on within coffer-dams, and during stages of the river from low water to 14 ft. above it. Owing to the permeable nature of the material, there has always been considerable percolation, which has required pumping to keep the pit sufficiently clear of water to enable construction to proceed. The water escaping from under the concrete already placed carries away the fine material directly under the concrete, leaving a space between it and the sand through which the transmission of pressure will be direct, and, consequently, any pressure which may be developed will be uniformly exerted over the whole base of the structure.

Mr. Oakes.

However interesting it may be to know what the upward pressure under a dam would be under ideal conditions (assumptions), such as tight sheeting, uniform material under dam, uniform weight on sand, etc., it must be borne in mind that such conditions will never be obtained throughout the length of the dam as actually constructed; that the computed pressure based on those assumptions will be the least possible pressure; and that the dam must be designed to withstand at all sections the greatest possible pressure, which will be, under ordinary circumstances, that due to the full hydrostatic head.

Mr. King.

H. W. KING,* M. AM. SOC. C. E. (by letter).—Mr. Colman deserves great credit for having made a careful and systematic study of an important engineering problem of which but little is known. The engineer is confronted with this problem whenever he undertakes the design of a dam, and especially a dam of the type commonly constructed on earth foundations. At present there seems to be no uniformity of practice among engineers in allowing for upward hydrostatic pressure. Cases might be cited in which no allowance was made for such a pressure, and in other cases allowance has been made for a pressure equal to that exerted by the total head under the entire base of the dam.

The point at which the resultant pressure under a dam should be applied is an important consideration, as is also the effect of sheet-piling on upward pressure and in reducing seepage. The author's experiments throw much light on all these important points.

The writer was in close touch with Mr. Colman during the progress of these experiments, and there is no doubt that each detail was worked out with great care and thoroughness. It does not seem probable that many more data could be secured from a series of experiments of this nature than were obtained by Mr. Colman. Some inconsistencies appear in the results, but, in a great measure, these may be explained, and do not necessarily detract seriously from the value of the experiments as a whole.

The criticism has been made that the scale of the experiments was too small, and that a larger model would give more reliable results. This may be true, but Mr. Colman's model was approximately 10 ft. high and 16 ft. long, and a much larger one would hardly be feasible. Experiments on a larger scale must be made on existing dams, and it should be possible to obtain much valuable information in this manner. Such data, however, must necessarily be incomplete, and experiments on models like Mr. Colman's will be valuable in interpreting and extending them.

One important point brought out by these experiments was that sheet-piling, to be effective in preventing upward pressure, must be

* Ann Arbor, Mich.

Mr. King. absolutely water-tight. The importance of this fact was not realized early in the investigation, and proper precautions were not taken with the first piling to prevent leakage. It was found that the effect of such piling could scarcely be noticed. This condition was changed immediately, however, when the piling was made impervious.

The effect of tight piling may be seen from Figs. 11 and 12. With no piling, for a depth of water of 5 ft. above the dam, the upward pressure head just below the dam is approximately 4 ft., and, with 1-ft. sheet-piling at the heel of the dam, it is 2.5 ft.

Unfortunately, the relation between length of piling and total upward pressure cannot be seen from these experiments. There can be no doubt that the upward pressure must decrease as the piling is lengthened, but, from Figs. 13 and 14, the opposite appears to be the case. This anomaly the writer believes is due to leaky sheet-piling. In spite of the fact that every apparent precaution was taken to prevent leakage, it is probable that some water was able to find its way through the piling, and thus in a measure nullify its effect.

This principle has an important practical bearing on dam construction. It is a common practice to drive a row of sheet-piling at the heel of a dam. There is no kind of sheet-piling that can be considered water-tight, especially when the usual difficulties encountered in driving are taken into consideration. It would appear, then, that such piling has little or no effect in reducing upward pressure under the dam. The writer is of the opinion that a good impervious concrete cut-off wall of moderate depth will have a more beneficial effect in this regard than any quantity of sheet-piling.

The experimental data on seepage appear to be a little conflicting. The writer believes, however, that a cut-off, to be effective in reducing seepage, must be water-tight and extend into an impervious stratum. Frequently, the seepage losses through the banks around a dam are a more important consideration than seepage under the dam.

The author submits a formula for determining the total upward pressure on the base of a dam. This formula has been shown in earlier discussions to be inapplicable in extreme cases. This is only to be expected, as the formula, which is derived from a small number of points, all in the lower limits, has no theoretical basis. No empirical formula of this kind can be used with real assurance beyond the range of the experimental data on which it is based.

It may be seen from Figs. 9 to 15, that, for any experiment, if a straight line is drawn from a point corresponding to the depth of water above the dam to the toe of the dam, this line will, in general, lie above the pressure line corresponding to this experiment. From this it appears that if the upward pressure on the base of the dam is taken as the hydrostatic pressure due to one-half of the head against

the dam, the assumption will be safe. It seems reasonable to assume that this rule should apply to higher heads and larger dams. Mr. King.

An examination of Table 5 shows that, with sheet-piling at the heel of the dam, the upward pressure is equal to about three-tenths of the total hydrostatic pressure. The experiments with 2-ft. and 3-ft. piling would doubtless show a total pressure less than this, if there had been no leakage through the sheet-piling. The author has pointed out the fact that an impervious cut-off at the toe of a dam will increase the upward pressure and move the point of application of the resultant pressure down stream.

Using the author's notation, in which H, L, and P represent, respectively, the head of water against the dam, the length of cross-section of the base of the dam, and the total upward pressure per unit length of dam, the following simple formulas should be safe guides in determining upward pressure:

(1) With no cut-off at the heel of the dam or with ordinary sheet-piling,

$$P = \frac{62.4}{2} H L = 31 H L;$$

(2) With an impervious cut-off at the heel of the dam,

$$P = \frac{62.4}{3} H L = 21 H L.$$

The point of application of the resultant, P, is $\frac{1}{3} L$ from the heel of the dam.

Both samples of sand used in these experiments were comparatively clean. It seems probable that a sand having the voids well filled with silt would show a smaller pressure. It would hardly seem safe, however, under any conditions, to assume a pressure smaller than that determined from these experiments, as earth in its natural state may not be of uniform mixture, and a seam of sand may be the controlling factor in producing pressure, rather than an average mixture of the material under the dam.

ROBERT E. HORTON,* M. AM. SOC. C. E. (by letter).—The writer had the pleasure of examining the apparatus and methods while this investigation was in progress, and has full confidence in the results, as far as they go. The data are well presented; the deductions are made in a logical and orderly manner; and the paper is an original and valuable contribution to experimental hydraulics. The writer fully realizes the labor and difficulties involved in prosecuting original investigations, and intends no criticism when he says that these results should be confirmed and extended by further studies. Mr. Horton.

* Albany, N. Y.

Mr. Horton. In view of the great present-day importance of hydraulics, in water-power, water supply, irrigation, drainage, sewerage, and navigation—matters vital to life and indispensable to commerce—efforts should be made to provide additional hydraulic laboratories and means to conduct experimental hydraulic and hydrological work therein along much broader lines than ever heretofore attempted. The average layman has a misty conception of the functions of a hydraulic laboratory; the average engineer probably looks on it as a place where neat little experiments are performed to illustrate to students the flow of water over weirs or through orifices under ideal conditions. That is a useful function, but much greater service may just now be rendered by providing means whereby life-size experiments on a wide variety of subjects may be carried out. There is hardly a subject in the domain of formal hydraulics that does not need a great deal of large-scale experimental work, and there are many subjects which have been scarcely touched experimentally; such, for example, as the phenomena of the standing wave. Furthermore, there are several subjects —in connection with dams, for example—in relation to which present-day design is based largely on very arbitrary assumptions.

The phenomena of the causes of floods, low-water flow of streams, yield of wells, evaporation, and many other hydrologic subjects could be investigated to great advantage in a good hydraulic laboratory, yet such laboratory studies have never been undertaken to any extent.

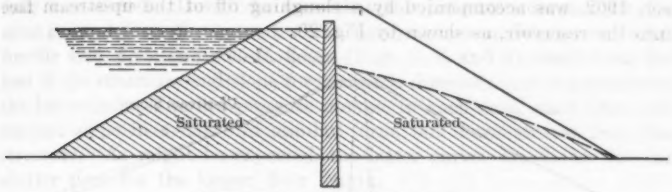
Mr. Colman's paper is a good example of a clean-cut and valuable investigation, capable of being carried out at moderate expense in a hydraulic laboratory. Even on this subject, however, more could have been accomplished almost in direct proportion to the time and means available. For example: experiments on dams with longer bases appear to be desirable; and valuable results could be obtained by conducting experiments on earth dam sections with cores.

The experimental results given suggest that, for an earth dam of homogeneous material with an impervious core, and on a permeable foundation, the water under the part of the dam down stream from the core would rise by static pressure and the down-stream portion of the dam might thus be saturated to the same, or nearly the same, degree if the core was absolutely impervious as if it was somewhat permeable. If, on the other hand, the dam is on an impervious foundation and the core is somewhat permeable, the water just below the core will stand at a height below that in the pond, dependent on the resistance to flow afforded by the core and the freedom of outflow of the percolating water. Apparently, the more permeable the material on the down-stream side of the core the less will be the degree of saturation of the down-stream face of the dam.

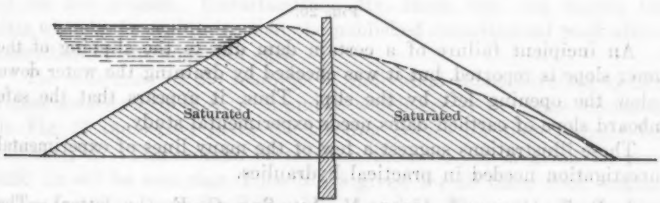
As regards conditions within the portion of an earthen dam up-stream from an impervious core, experimental work is also needed

Mr. Horton.

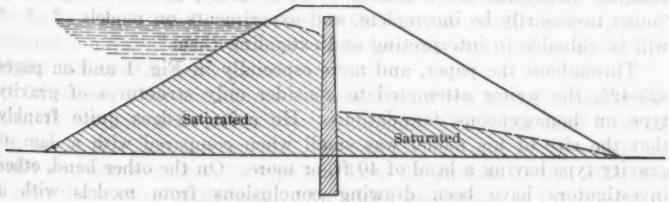
HYPOTHETICAL CONDITIONS WITHIN EARTHEN DAMS.



A.-PERMEABLE FOUNDATION WITH IMPERVIOUS CORE



B.-NON-PERMEABLE FOUNDATION, SEMI-PERMEABLE CORE. DOWN-STREAM SIDE COMPACT.



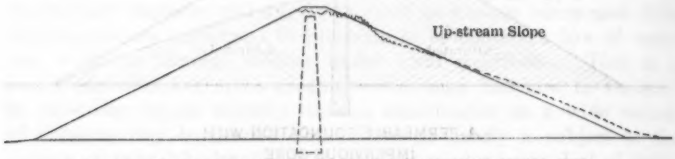
C. NON-PERMEABLE FOUNDATION SEMI-PERMEABLE CORE. DOWN-STREAM SIDE OF OPEN TEXTURE.

FIG. 19.

Mr. Horton.

to determine the best materials and their stability when saturated. These hypothetical conditions under earth dams are illustrated by Fig. 19. Confirmatory experiments are needed.

The failure of the Savage Reservoir, near Utica, N. Y., in September, 1902, was accompanied by a sloughing off of the up-stream face into the reservoir, as shown by Fig. 20.



CROSS-SECTION OF SAVAGE RESERVOIR DAM, NEAR UTICA, N.Y., SHOWING RELATIVE POSITION OF UP-STREAM SLOPE BEFORE AND AFTER FAILURE.

FIG. 20.

An incipient failure of a certain dam due to the slipping of the inner slope is reported, but it was checked by draining the water down below the opening left by the slip. Thus, it appears that the safe inboard slope of earthen dams needs experimental study.

These illustrations suggest a few of the many lines of experimental investigation needed in practical hydraulics.

Mr. Colman.

J. B. T. COLMAN,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer has read the discussions with much interest. As a result of the radically different opinions therein expressed, his own ideas have been made more lucid. A small part of the discussion is not closely allied to the subject.

At the present time, very few data pertaining to the hydraulics of existing structures are available. "Such data", as Mr. King states, "must necessarily be incomplete, and experiments on models * * * will be valuable in interpreting and extending them."

Throughout the paper, and more especially in Fig. 1 and on pages 423-425, the writer attempted to consider only structures of gravity type on homogeneous foundations. He acknowledges quite frankly that the size of his model was small when compared with a dam of gravity type having a head of 40 ft. or more. On the other hand, other investigators have been drawing conclusions from models with a capacity ranging from less than 1 cu. ft. up to 16 cu. ft. This model was approximately 16 ft. long, 4 ft. wide, and 10 ft. high.

The sands used were very uniform in analysis. Several samples, taken while removing them from the model, and afterward analyzed,

* Detroit, Mich.

showed almost identical results for each sand. At the same time, no indication of piping action was observed. Mr. Colman.

Before taking up the experiments, the ratio of the area exposed to pressure, the width of the dam, and the area at the toe, were carefully considered. It will be seen that the pressure curves show the same general form throughout. The high-pressure curves, in each case, for the 4.25, 6.25, and 8.25-ft. floors (Figs. 2, 3, and 4), start from the heel of the structure and slope forward in a direction that approximates the horizontal. The low-pressure curves, in each case, start from the toe and slope in a curve of similar form, but down stream from the structure. As might be expected, the latter curves are flatter for the shorter than for the longer floor length.

Again, attention must be called to the fact that there are no readings which would indicate a restricted entrance or exit. That which appears at first to be the result of insufficient exposed surface at the heel and toe of the structure is, as a matter of fact, the indication of the law sought. Unfortunately, Mr. Hays does not supply the data whereby he maintains that the published experimental work shows restriction of entrance and exit.

The difference in the readings of individual tubes is raised by Mr. Oakes, as shown by Nos. 2, 5, and 8, which are at the same elevation. In Fig. 21 the writer gives the leveling curves for these three tubes for Sand *B*. The method of plotting these curves is given on page 429. It will be seen that Tube No. 8 gives, for all three floor lengths, values less than those for Tubes Nos. 2 and 5. The same tube leveling curves for Sand *A*, with a single exception for Tube No. 2, hold the same relative position. That is, the lowest for all three floor lengths is Tube No. 8, then Tube No. 5; and the highest is Tube No. 2.

Throughout his work, the writer strove to forget the mathematical demonstration, as indicated by Mr. Muckleston, and given more fully by Parker* and others. This proof is very interesting, but is based on assumptions purely theoretical. One hardly needs to go into a discussion to show the weakness of a formula obtained as the result of the indicated operations.

Of course, when one admits the weakness of the mathematical equation, the objection to the tail-water standing above the sand level at the toe of the dam is removed. In practice, no one would expect to build a structure of gravity type and keep the surface of the tail-water just at the surface of the sand. An added precaution in this direction would rather detract from than add to the usefulness of the data.

The writer is very much gratified indeed that other investigators, working along similar lines, have obtained results which agree wholly

* "The Control of Water", by Philip & Morley Parker, Assoc. M. Am. Soc. C. E., pp. 279–299.

Mr. Colman.

or in part with Conclusions 1, 2, 3, and 4. It would seem that the partial disagreement of Mr. Hays indicates that he has worked with a small number of data, or with too few piezometer tubes.

It is hardly necessary to explain why the law of pressure, between certain limits, should resemble closely the curve of the law of error. In fact, there is no satisfactory mathematical proof that applies to the law of error itself. The point to be considered is that the writer, in attempting to determine a curve which would most closely fit the cases in hand, could find none that would approximate all points so nicely as the probability curve. It is hoped that other investigators, with more data at hand, may be able to determine a simpler curve, which can be solved directly.

Mr. Elliott and Mr. Creager state very correctly that, if the values of $\frac{H}{L}$ are increased to 1.75 and 1.4, respectively, the equation gives greater values than it is possible to obtain. It would seem to the writer that, for the time being, these two gentlemen lost sight of the fact that gravity-type dams on porous foundations are rarely constructed with a ratio of H to L greater than 1 to 2, and more often with a ratio of 1 to 5.*

In this instance, the curve was extended from three to seven times its practical limit. Judging from Mr. Creager's curve in Fig. 18, it would seem that the formula is not at fault for values of S less than unity. The writer was fully aware that his formula would not hold for structures on so-called impervious or semi-impervious foundations, into which classes structures with values of S greater than unity fall.

Mr. Oakes brings up the same question in a different form when he says, "For a 5-ft. head, the pressure at the heel, with bases 4.25 and 8.25 ft. long, is greater than that for the base 6.25 ft. long". It is very obvious that the pressure at all points is a function of the slope, that is, of both H and L. The 5-ft. head with the 4.25-ft. base is, as pointed out, above the working value of S. Under such a structure, if the action was continued, piping action would doubtless take place and later failure. The material difference in the percentage of pressure, Fig. 9, for the 3 and 4-ft. heads over the differences in percentage of pressure for the 1, 2, and 3-ft. heads, indicates the approach to the limit of the working value of S.

The writer purposely avoided the question of the effect of the sand coefficients and sheet-piling until he had disposed of the items of fundamental importance. Not that the effect of these coefficients is unimportant; but that, in experimental work, it follows rather than precedes the above.

* The reader is referred to the paper entitled "Dams on Sand Foundations: Some Principles Involved in Their Design, and the Law Governing the Depth of Penetration Required for Sheet-Piling", by Arnold C. Koenig, Assoc. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXIII, p. 175.

Mr. Colman.

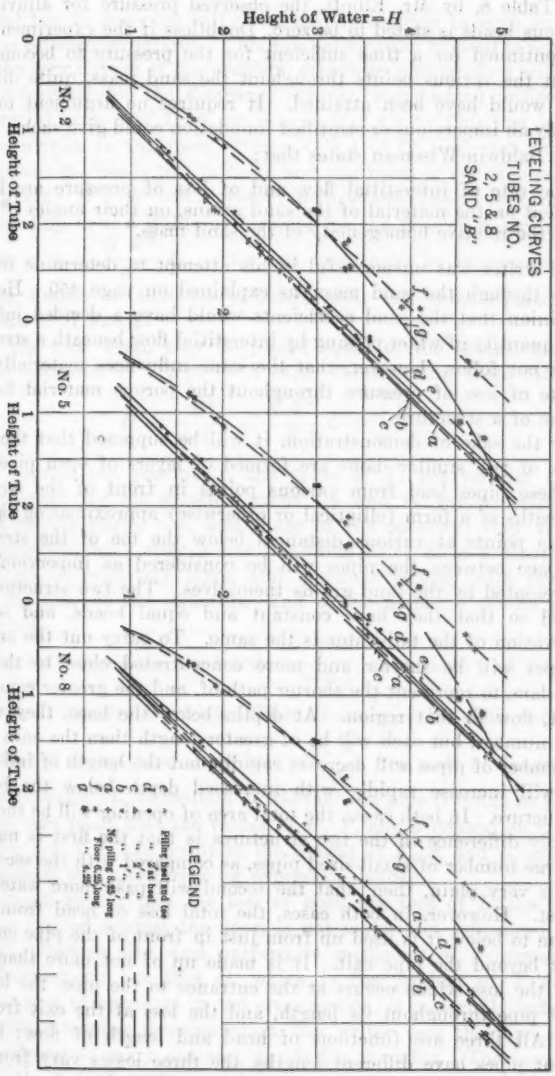


FIG. 21.

Mr. Colman.

In Table 8, by Mr. Elliott, the observed pressure for alluvial soil at various heads is stated to be zero. Doubtless if the experiments had been continued for a time sufficient for the pressure to become constant at the various points throughout the sand mass, quite different results would have been attained. It requires no argument to show that only an impervious or stratified foundation could give such a result.

Mr. Baldwin-Wiseman states that:

"The rate of interstitial flow and of loss of pressure are largely dependent on the material of the sand grains, on their angles * * * and on the relative homogeneity of the sand mass."

The writer was unsuccessful in his attempt to determine the rate of flow through the sand mass, as explained on page 450. He is of the opinion that the sand coefficients would have a decided influence on the quantity of water passing by interstitial flow beneath a structure. It does not follow, however, that the same influences materially alter the rate of loss of pressure throughout the porous material forming the base of a structure.

For the sake of demonstration, it will be supposed that the foundations of two similar dams are formed of layers of open pipes, and that these pipes lead from various points in front of the structure along paths of a form (elliptical or otherwise) approximating the flow lines, to points at various distances below the toe of the structure. The space between the pipes will be considered as impervious, and is represented by the sand grains themselves. The two structures are situated so that they have constant and equal heads, and so that the elevation of the tail-water is the same. To carry out the analogy, the pipes will be shorter and more concentrated close to the floor of the dam, to represent the shorter path of, and the greater concentration of, flow in that region. At depths below the base, they will be less in number, but each will be of greater length than the ones above. The number of pipes will decrease rapidly, but the length of individual pipes will increase rapidly with increased depth below the base of the structure. In both cases, the total area of opening will be the same. The only difference in the two structures is that the first is made up of a large number of small-sized pipes, as compared with the second.

It is very plain, then, that the second will pass more water than the first. However, in both cases, the total loss of head from above the dam to below it is used up from just in front of the pipe entrance to just beyond the pipe exit. It is made up of not more than three losses: the loss which occurs at the entrance to the pipe, the loss per foot of pipe throughout its length, and the loss at the exit from the pipe. All three are functions of head and length of flow; but, as different pipes have different lengths, the three losses vary from pipe to pipe. The velocity of flow in each pipe will adjust itself so that

the total loss in one pipe will equal that in another in each structure, and also between structures. This total absorption of head is necessary, from the very nature of the problem, otherwise piping effect will occur. Mr. Colman.

The formula for the discharge through sharp-edged orifices is of the form, $Q = m\,A\,\sqrt{2\,g\,h}$, or $V = m\,\sqrt{2\,g\,h}$.* This last equation may be written in the form, $V = m\,\sqrt{2\,g}\,\sqrt{H' - h''}$, where H' and h'' represent the heads just in front, and just back, of the pipe entrance, respectively. $H' - h''$ will represent the head consumed in entrance loss.

According to the Williams formula for the flow of water in pipes, the loss is $h_f = \frac{K\,V^n}{d^0}\,L$. The exit loss could be represented by a formula similar to that for entrance loss. All three equations are functions of H and L.

H, the total head, may be placed equal to the several losses, or $H = h$ entrance $+$ h pipe $+$ h exit. Substituting these values, we have:

$$H = \frac{V^2}{m\,2\,g} + \frac{K\,V^n}{d^0}\,L + \frac{V^2}{m'\,2\,g}.$$

As pointed out previously, the value of V, the velocity of flow, is very small for all practical cases; m and m' are constants, and depend on the angle, size, and shape of the opening; they vary but little, however, with size; g has the value 32.2; K is a constant; L, the length, is relatively large, and increases rapidly with the depth below the floor of the structure; and d, the diameter of the pipe, is very small.

Now, as V decreases, the fraction, $\frac{V^2}{m\,2\,g}$, decreases; and for small values of V, it approximates 0. Then $H = \frac{K\,V^n}{d^0}\,L$. This expression states that H is expended at the rate of $\frac{K\,V^n}{d^0}$ per unit of length over the entire length, L, regardless of whether the pipe is rough or smooth.

It is very plain that the values of K, V, and d depend on the sand analysis; but V is the result of the weight of K, d, and L, and takes a value such that the foregoing expression holds.

It would appear that the loss of head per foot of L, the upward pressure exerted against the floor, is not materially influenced by the porosity, effective size, and uniformity coefficient of a homogeneous sand foundation; but that Q, which is the product of V and the effective area of openings in the cross-section, is materially influenced.

In the foregoing demonstration, the writer uses the pipe analogy in an attempt to carry out and clarify his thoughts. The comparison

* Chapter 6, "Mechanics of Engineering," by Irving P. Church, Assoc. Am. Soc. C. E.

Mr. Colman.

has been briefly referred to by different writers, none of whom, however, to his knowledge, carries it through to a conclusion.

Attention might be called to the fact that this analogy is not quite parallel, in that the effective area of openings in the sand increases somewhat from the surface of the sand in the bottom of the reservoir to some point beyond the toe of the dam. This influence, then, would alter the value of V throughout the length of its course; it would increase its value at the entrance, cause it gradually to decrease throughout the length of flow, and decrease it at the exit. In the foregoing equation, the entrance loss would be increased, the loss of head per foot of pipe would have a gradually decreasing value throughout the length, L, and the exit loss would be decreased. This small secondary influence of a greater or less change of velocity, as between structures, can alone act to alter the form of the upward pressure curve.

Mr. Creager and others think that for a sand equally pervious at all points, the upward pressure on the base of a dam without cut-off approximates $\frac{H}{2}$, irrespective of the size or ratio of the head to the base length. Designers have used values for upward pressure ranging from zero to H over the entire base length, L. They also disagree as to the point of application of this upward pressure. Some have used $\frac{L}{3}$ as the distance from the heel to its center of gravity, and others have used values as high as $\frac{L}{3}$ from the toe. It would seem, the plotted curves and data make this point clear, as Professor King points out, that the upward pressure is somewhat less than $\frac{H\ L}{2}$, or:

$$P = 31\ H\ L.$$

It also shows that the point of application of this force is $\frac{L}{3}$ measured from the heel of the dam.

Turning to the condition of sheet-piling, the writer feels justified in stating that, in the case of Sand *A*, he had used reasonable precautions in placing the 1-in. lumber piling at the heel. The results were unsatisfactory. With Sand *B*, a single piece of sheet metal was in each case closely tacked to the box on the two sides and to the heel of the dam to form the sheet-piling.

It would seem that, as Mr. Travell points out, "As the sheet-piling was made longer, a decrease in pressure would naturally be expected". If the indicated data are influenced so markedly, as they appear to be, by very small leaks, surely it emphasizes the desirability of using an

impervious cut-off wall instead of sheet-piling at the heel of the structure. Mr. Colman.

It is not suggested that the use of an impervious, comparatively shallow, cut-off wall will reduce the flow, as some of those who have discussed the paper seem to assume; but that it will greatly reduce the upward pressure on the floor. The safety of a structure does not depend on the quantity of water that may pass through the sand foundation, but on the velocity and pressure head that may reach the exit layers of sand and wash away those particles.

Such a cut-off wall would concentrate the flow lines at its bottom, where no piping or washing action could take place. These lines, after passing the base of the cut-off, would spread out fan-like through an angle approximating a quadrant. This action would mean a rapidly increasing area of effective cross-section and a correspondingly reduced velocity of flow, with the accompanying loss of head. By altering the direction of flow, its concentration, and the resulting velocity of flow and loss of pressure head, where no removal of particles can occur, the upward pressure on the base of the dam is materially decreased, and its safety is proportionately increased.

The writer wishes to emphasize the point that an impervious cut-off wall of moderate depth at the heel of a structure will have more beneficial results than any quantity of sheet-piling.

All seem to agree that sheet-piling at the toe of a structure, if used, should be loosely driven.

Mr. Wegmann cites the failure of the dam at Austin, Pa., to refute the use of a cut-off wall, such as just outlined. The cross-section of this structure was of such a form as might be used on any moderately sound rock foundation, and was not designed for a porous foundation. It seems to the writer that the failure of this structure, the foundations of which were of stratified shale and sandstone, does not reflect on the point at hand.

The writer firmly believes that the effect of ordinary driven sheet-piling, either of wood or steel, as a cut-off for pressure head, is practically zero. The use of long-length, steel, sheet-piling and anchor-piles, similar to those cited by Mr. Oakes, in a low diversion weir, has come under his observation. In this case (the original head weir on the Tarlac Irrigation Project, Tarlac, Philippine Islands), with the foundation and weir complete, and with a head of perhaps one-fifth of that for which the structure was designed, piping action was going on. Needless to say, when a flood came, shortly afterward, the structure failed.

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Paper No 1357

THE TWELFTH STREET TRAFFICWAY VIADUCT, KANSAS CITY, MISSOURI*

By E. E. Howard, M. Am. Soc. C. E.

With Discussion by Messrs. F. W. Green, L. J. Mensch, Howard W. Holmes, M. M. Upson, E. A. Slettum, L. R. Ash, John Lyle Harrington, and E. E. Howard.

Synopsis.

This paper describes the planning, the design, the equipment for, and methods of, construction, of a double-deck, reinforced concrete, trafficway viaduct, 2 300 ft. long, 60 ft. wide, and 120 ft. high, recently opened for traffic in Kansas City, Mo. Some special applications of general theories involved in the calculations for design are given, and summarized and classified costs are included. The features deemed of particular interest are the design of a large arch forming a portion of the structure, and the arrangement of the contractor's plant and equipment.

For convenience the paper is divided into the following parts:

1.—Introductory and historical.
2.—General description of structure.
3.—Loadings, specifications for, and methods of, design.
A. Investigation of columns.
B. Design of arch.
4.—Specifications for materials and workmanship.
5.—Equipment for, and methods of, construction.
6.—Quantities and costs.
7.—Personnel.

* Presented at the meeting of September 1st, 1915.

The description of features and details which may be grasped readily from an inspection of the accompanying illustrations is omitted.

1.—Introductory and Historical.

The Twelfth Street Trafficway Viaduct forms the principal part of the Twelfth Street Trafficway, a general municipal improvement designed to facilitate usual street traffic between sections of Kansas City, Mo., which are separated topographically. The total cost of the whole improvement, including land damages, was about $1 000 000, the viaduct structure costing about $650 000. The funds were provided by a portion of a general bond issue of $475 000, by assessments against a benefit district of special tax bills of $325 000, and by a contribution of $200 000 from the local street-car company in return for operating privileges over the structure.

The rather flat, level valley of the Kaw River, in Kansas City, locally known as the "West Bottoms", is about 1 mile wide, and is occupied by railroad yards, freight stations, warehouses, packing-houses, wholesale and manufacturing concerns, and the large stock yards interests, including a considerable percentage of the city's business. It lies at an elevation some 150 ft. below the principal residential, retail, and office building districts of the city, the break between the two elevations being rather precipitous bluffs and hills from 100 to 200 ft. high. Twelfth Street is the principal east-and-west thoroughfare of the city, and may be said to be a central axis, both with respect to the up-town part of the city, and the industries in the Bottoms. There have been only three routes for vehicular travel between these levels: the northern routes, requiring a diversion of about ½ mile north of Twelfth Street, and a southern route, requiring a similar diversion of more than 1 mile south. About thirty years ago, a steel viaduct for cable cars was built on Twelfth Street with a grade of about 13%, and car service was maintained until the present construction was commenced.

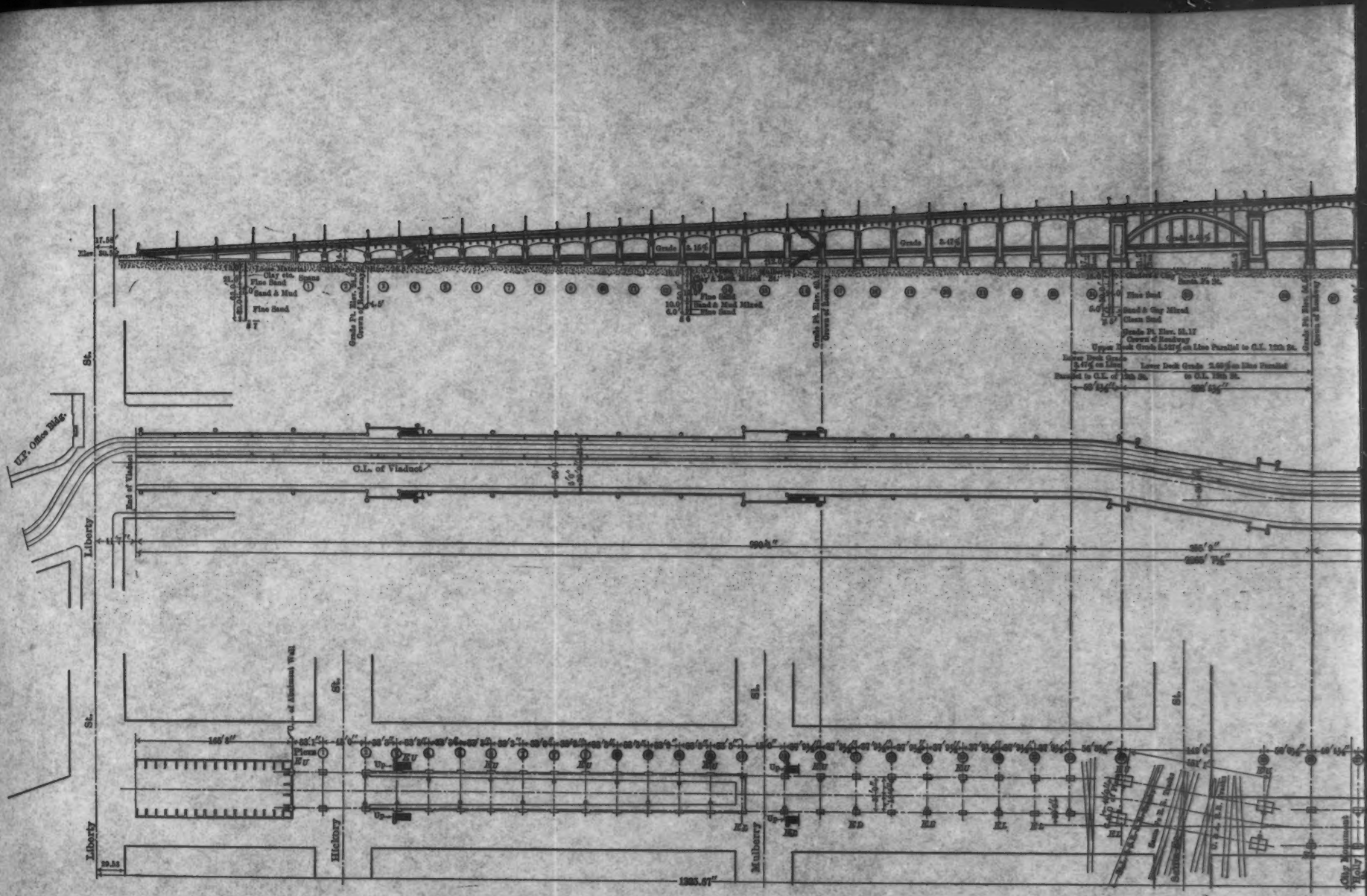
A roadway suitable for vehicular, as well as street-car, traffic on a viaduct to the top of the hill, on a viaduct to the bluffs, and in a tunnel emerging at some convenient point, and other such arrangements, have been advocated for many years, and have been made a feature of more than one political campaign, and of franchise negotiations with street-railway interests. The improvement finally realized as the Trafficway

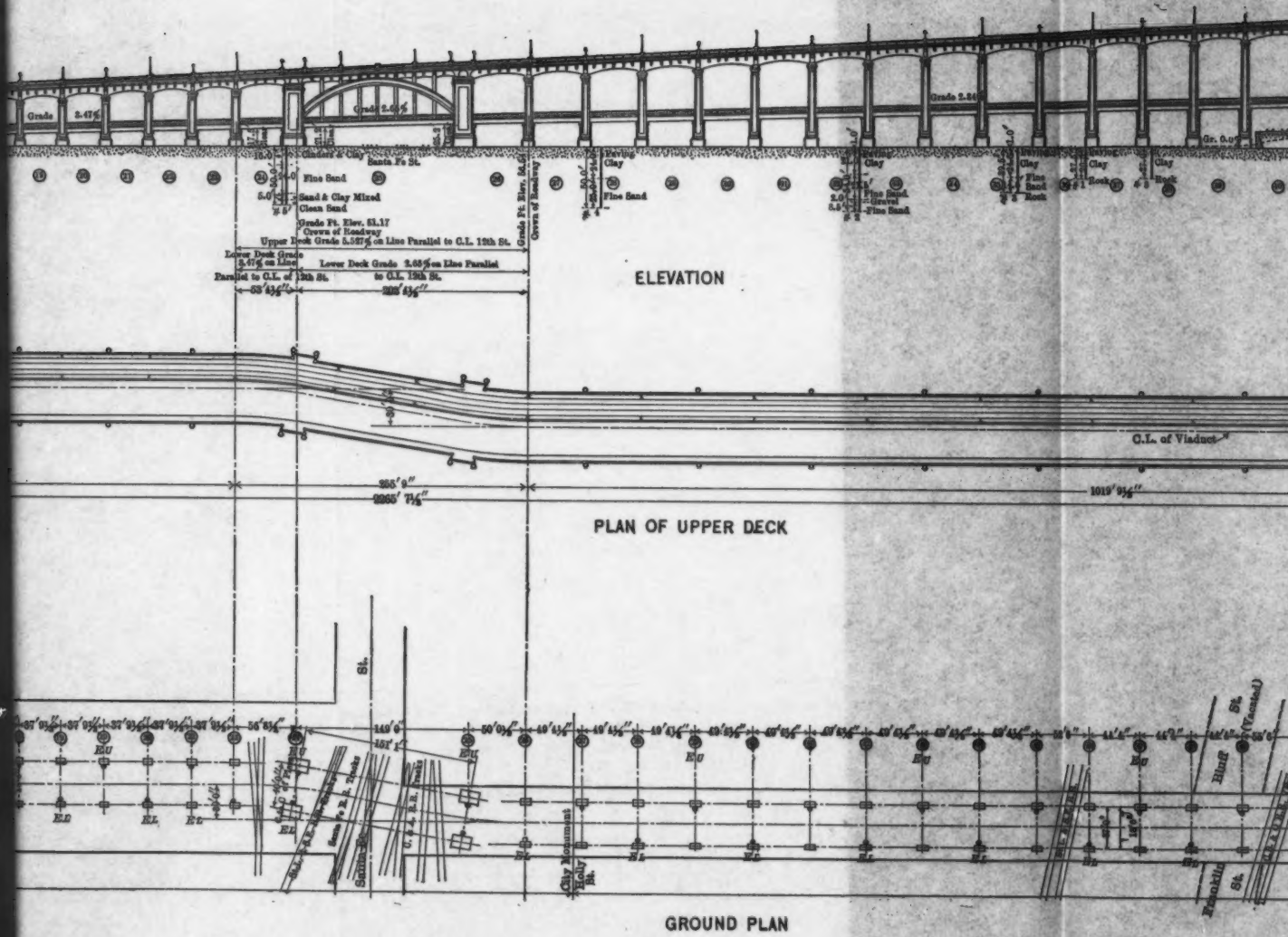
extends from Liberty Street to Broadway, a distance of ¾ mile, of which the viaduct occupies about 2 300 ft. The remaining portion consists of earth embankments and cuts, with the usual street improvements on Twelfth Street, and the regrading of certain side streets to intersect its new grade. A street 60 ft. wide is thus provided on a continuous grade of about 5.5% for a distance of 3 500 ft., substantially from end to end of the improvement. It was physically impossible, without involving prohibitory expense and damage to existing structures, to extend the end of the grade in either direction, so the gradient practically fixed itself. To accommodate traffic desiring a less steep grade, and willing, for such advantage, to travel by a less direct route, a roadway on a lower deck is provided, with a grade of about 2.5 per cent. This lower roadway begins one block east of the upper one and extends to the bluff, a distance of 1 800 ft.; thus enabling all classes of traffic to avoid the grade crossings over the heavy service railway tracks on the street below. Ascending roadways or streets alongside the bluff from the end of the lower deck will terminate in streets at the summit of the hill, giving easy grades, although by indirect routes for team traffic bound up town. These connecting streets are not yet built.

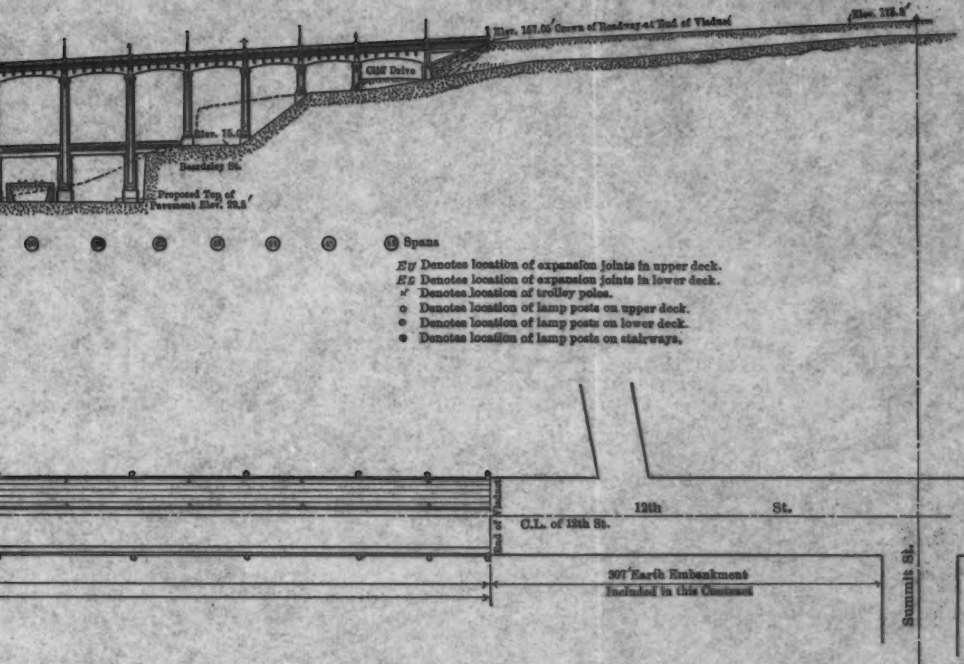
Preliminary studies and estimates were made for viaducts of steel and of reinforced concrete; in the latter case, structures of arch spans, and also of girder spans, were considered. The city officials were favorable to the engineers' recommendations of concrete, and the girder type of structure was selected, the final general plan being as shown on Plate XVI. The suspended lower deck over Santa Fé Street, of the final plan, was developed instead of short spans with intermediate columns, to meet the desire of the railways for a clear span the full width of the street.

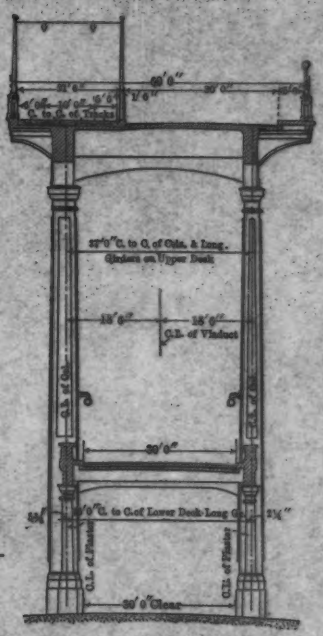
2.—General Description of Structure.

The Twelfth Street Trafficway Viaduct is a reinforced concrete structure consisting of girder spans supported by columns founded in part on rock, in part on soil, and in the main part on concrete piles. Over eight railway tracks in Santa Fé Street a long span is provided by the construction of an arch. The structure is about 2 300 ft. long, 60 ft. wide, and 120 ft. high at its highest point. The columns are in pairs transversely, and are placed so that the upper deck, which has a total width of 60 ft., cantilevers beyond the columns on each side, and


Grade 5.15%
Grade 5.47%
Fine Sand
Sand & Mud
Fine Sand
Fine Sand
Sand & Mud Mixed.
Fine Sand
Fine Sand
Sand & Clay Mixed
Clean Sand
Grade Pt. Elev. 31.17
Center of Roadway
Upper Deck Grade 5.527% on Line Parallel to C.L. 12th St.
Lower Deck Grade 5.65% on Line Parallel to C.L. of 12th St.
Lower Deck Grade 2.65% on Line Parallel to C.L. 12th St.
C.L. of Viaduct
U.P. Office Bldg.
Liberty
St.
Hickory
Mulberry
Abutment Wall
Piers
Up
Down
Elev. 30.5
Elev. 31.17
Grade Pt. Elev. 31.17
Center of Roadway

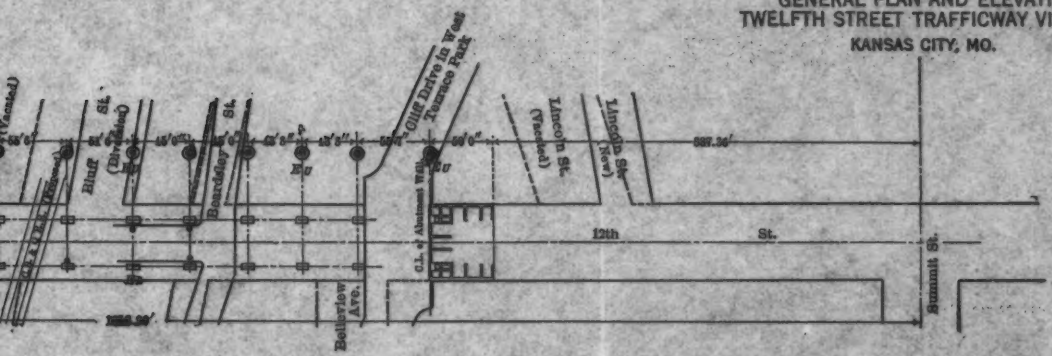

Grade 2.47%
Grade 2.64%
Clay & Clay
Santa Fe St.
Fine Sand
Sand & Clay Mixed
Clean Sand
Grade Ft. Elev. 51.17
Crown of Roadway
Upper Deck Grade 5.587% on Line Parallel to C.L. 19th St.
Lower Deck Grade 2.47% on Line Parallel to C.L. of 19th St.
Lower Deck Grade 2.65% on Line Parallel to C.L. 19th St.
53' 4½"
225' 4½"
Grade Ft. Elev. 54.0
Crown of Roadway
Perry Clay
Fine Sand
Clay
Fine Sand
Gravel
Fine Sand
Clay
Fine Sand
Rock
Clay
Rock
Gr. 0.0%
ELEVATION
C.L. of Viaduct
355' 9"
2265' 7½"
1019' 9½"
PLAN OF UPPER DECK
St.
Santa Fe R.R. Tracks
U. P. & S. P. R.R. Tracks
Clay Monument
Holly St.
Main St.
Franklin St.
GROUND PLAN





TYPICAL CROSS-SECTION

GENERAL PLAN AND ELEVATION
TWELFTH STREET TRAFFICWAY VIADUCT,
KANSAS CITY, MO.



the lower deck roadway passes between them. The upper deck provides a roadway 30 ft. wide paved with creosoted blocks, a sidewalk 5 ft. wide, and a street-car space 22 ft. wide. The roadway is separated from the street-car space by a concrete curb, and on each side of the structure there is a concrete hand-rail. The two street-car tracks are built with the usual wooden cross-ties set in ballast, so that no other traffic can use the area which they occupy. This arrangement, somewhat uneconomical from a highway standpoint, was specified by the city authorities. Iron trolley poles on each side of the street-car space support the usual overhead trolleys. Stairways are provided at Hickory and Mulberry Streets to give access for pedestrians to the roadway from the ground surface. The lower deck, beginning at Hickory Street and ending at Beardsley Street, provides a single roadway, 30 ft. wide, paved with creosoted blocks. The longitudinal girders supporting this deck extend far enough above the roadway to form side barriers.

The upper deck comprises forty-five deck-girder spans of two girders each, varying in length from 33 to about 56 ft., the arch span, and two earth-filled approaches. The lower deck comprises twenty-seven through-girder spans of two girders each, supported on the same columns that carry the upper deck, the suspended deck of the arch span, and the earth-filled approaches. The floor-slabs are supported on cross-girders and cantilever beams. For the crossings of both decks over the streets, near the west end, very shallow floors are necessary, and concrete-encased steel beams in the floors, and concrete-covered steel girders, are used instead of the ordinary reinforcement. Both upper and lower roadways are lighted with incandescent electric lights, placed above the hand-rail for the upper deck, and on brackets on columns for the lower deck.

Below the structure Twelfth Street is about level from Santa Fé Street to the hillside, and gives access to Bluff Street leading northward along the bottom of the bluff. The space transversely between columns at the street level is 30 ft. Near the bluff, passing under the lower deck, the Kansas City Terminal Railway Company has two tracks carried by a concrete girder span above the ground surface of Twelfth Street, making four traffic levels at this point. Higher up on the hillside, just in front of the east abutment, the Kersey Coates Drive of the Park System passes below the upper-deck level.

Twelfth Street from Broadway to Santa Fé Street is 60 ft. wide, but for the three blocks, from Santa Fé Street to Liberty Street, it is only 30 ft. wide. Eleventh Street for these three blocks is 60 ft. wide, and the blocks between Twelfth Street and Eleventh Street are only 40 ft. wide. By securing these three very narrow blocks of property, a single street, 130 ft. wide for a distance of three blocks, was obtained, and the viaduct was placed about midway between, affording an excellent approach and retaining both Twelfth Street and Eleventh Street. This caused a shift in the alignment of about 30 ft., made by two easy curves at Santa Fé Street.

Careful attention was given to the architectural treatment in an effort to secure something more than a plain series of posts and beams. The treatment developed considers the columns as columns with plinths and capitals, and not merely as posts. The bottoms of the upper-deck girders are curved to give an arched or high cambered appearance. The lower-deck girders are straight and in effect are supported by secondary pilasters set out from the main columns. Certain limiting conditions of the railway and street locations crossing under the structure practically determined certain column locations, but the span lengths were made smaller near the lower end of the structure, affording uniformity of appearance and preserving the unity of effect, in spite of the great variation in height of the different columns.

The large arch at Santa Fé Street provides a central feature for the structure and was thus developed. The piers of the arch are carried up to the upper roadway and there support, on cantilevers, bays projecting beyond the usual hand-rail line. The capitals, plinths, mouldings, the projecting panels of the columns, and such features, are bold and heavy, without excess of fine lines, in keeping with the general dimensions of the structure. The architectural appearance has been favorably commented on, and it is believed effects a satisfactory solution of a rather difficult problem of a straight-line structure to be built with all possible economy and without any extra funds for adornment.

3.—Loadings, Specifications for, and Methods of, Design.

Assumed Live Loads.—The following loadings were assumed for design:

1.—For Slabs and Floor Systems:

On street-car tracks, Class 25, consisting of two coupled cars, each 52 ft. long and each weighing 50 tons, with impact allowance loads consisting of percentages of the preceding static loads, determined by the formula,

$$I = \frac{200}{L + 270},$$

where L is the length of track loaded;

On roadways, Class A, with an added impact allowance percentage determined by the formula,

$$I = \frac{100}{L + 150},$$

which, in total, consists of a load varying from 147 lb. per sq. ft., for a loaded length of 100 ft., to 200 lb. per sq. ft.;

On the sidewalk, Class B and impact, a corresponding loading varying from 123 to 167 lb. per sq. ft.

2.—For Girders and Arch Ribs:

On street-car tracks, Class 20, consisting of two coupled 40-ton cars, with impact percentage added;

On the roadways, Class B with impact;

On the sidewalk, Class C with impact, consisting of a load varying from 98 to 134 lb. per sq. ft.

The roadways were also considered to carry a 15-ton road roller, but without impact.

The wind loads figured were 30 lb. per sq. ft. of exposed surface for the unloaded structure, and 15 lb. per sq. ft. of exposed surface for the loaded structure. When considering maximum combined conditions of wind and temperature stresses, the allowable stresses were increased 30%, or as described later.

Allowed Unit Stresses.—For these assumed loadings, the following unit stresses were allowed in the design:

"For steel in tension, 15 000 lb. per sq. in.; for concrete in bending compression, 600 lb. per sq. in., tension zero; for concrete in direct compression, 450 lb. per sq. in., with not more than 400 lb. per sq. in. under expansion plates; for concrete in direct shear, 150 lb. per sq. in.; for concrete in diagonal tension, 35 lb. per sq. in.; for concrete in diagonal tension with part of steel bent up, 50 lb. per sq. in.; for concrete in diagonal tension with full shear reinforcement, 100 lb. per sq. in.; adhesion between concrete and deformed bar, 100 lb. per sq. in."

Designing Rules.—Some of the general rules for proportioning were:

"Column reinforcement shall have hoops spaced not farther apart than fifteen diameters of the vertical bars. The vertical steel shall be not less than 0.8%, nor more than 2.0%, of the minimum section of the column. The ratio of moduli of elasticity shall be taken as 15. The span length shall be taken as the distance from center to center of supports, but not to exceed the clear openings plus the depth of the slab or beam. The width of T-beams shall be considered to be not more than one-fourth of the span. Tension bars shall be spaced apart not less than three diameters of the bar, nor more than the thickness of the slab. If bond stress exceeds 100 lb. per sq. in., the ends of the bars shall be bent into a hook. The distance of the center line of bars to the surface of the concrete shall be not less than 1.3 d plus 0.17 in.; or, say, $\frac{3}{4}$ in. for $\frac{3}{8}$-in. bars, 1 in. for $\frac{1}{2}$ and $\frac{5}{8}$-in. bars, $1\frac{1}{2}$ in. for $\frac{7}{8}$ and 1-in. bars, and $1\frac{3}{4}$ in. for $1\frac{1}{8}$ and $1\frac{1}{4}$-in. bars. When stirrups are bent around the bars, allow an extra distance equal to the thickness of the stirrup, when the above distance is 1 in. or less, and equal to one-half the thickness when the above distance is more than 1 inch."

Controlling Conditions and General Methods.—The necessities of the surface street under the structure, from Santa Fé Street east to the bluff, required a transverse spacing of columns which placed one column almost midway under the street-car tracks and the other under the roadway. Thus, the columns, girders, and other parts of one side of the viaduct are much larger and heavier than those of the other side.

All parts of the structure, which could be so constructed, are designed for continuous action. The upper girders were figured as continuous beams for the various combinations of span lengths, assuming a uniform moment of inertia, a condition substantially existing; and the various sections of the girders were proportioned for the figured moments. The lower-deck girders were considered as beams restrained at their ends; the end sections were proportioned for the moment of truly fixed beams, and the center sections for a moment 10% in excess of the actual figured moment for the same condition.

The cross-girders and cantilevers of the upper deck were designed for the usual continuous conditions existing, and the lower-deck cross-girders as simple beams between the supporting girders. Considering longitudinal girders alone, it is easily apparent that the great number of spans of different length, and the varying combinations and num-

bers of spans between expansion points, required rather voluminous calculations for design. It may be of interest to state that the calculations for this structure cover about 300 closely-written, 10 by 16-in. sheets.

Two features of special interest, developed in the design, are the investigation of the columns and the calculation of the arch span at Santa Fé Street.

A.—Investigation of Columns.

The great height of part of the structure and the omission of all diagonal bracing in vertical planes, either transversely or longitudinally, made necessary a thorough investigation of the stresses due not only to every combination of live loads, to wind loads, and to changes in temperature, but especially to the secondary stresses from these loads, due to the rigid connections between the columns and the upper-deck and lower-deck cross-beams and longitudinal girders. It was clearly unnecessary to go to the extreme of attempting to design every separate column by calculating its every possible stress; so that a typical series of columns fairly representing the average was selected, and calculations, as nearly complete as might be, were made for them, in order to ascertain the maximum unit stresses that could be allowed for the direct loads as a simplified basis for column design, which would give results satisfactorily accurate. The essential features of this development are here presented in abbreviated form.

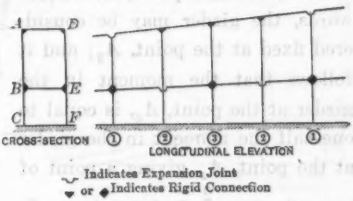


FIG. 1.

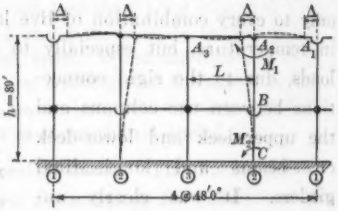
Fig. 1 shows a typical longitudinal section and cross-section of the viaduct. It will be noted that expansion joints are placed at every other column for the lower deck, thus eliminating all temperature stresses in columns, due to the shortening or lengthening of the lower deck. The upper deck has expansion joints generally at every fourth column, these joints staggering with those of the lower deck; all expansion joints for both decks are what may be called "double" expansion joints, the girders on both sides of the column being on sliding plates. The columns numbered (2) are, therefore, the only ones which have

stresses due to temperature changes. A complete investigation of this bent, therefore, was carried through.

To simplify the calculations, it will be assumed that all bents are of equal height and that the dimensions are as shown on Fig. 2.

Under the influence of a fall in temperature, the tops of Columns 2 will move toward the top of Column 3 equal distances, which are called Δ. Assuming that the lower-deck girders rest on frictionless rollers at the points, B, Columns 2 will deflect as if the lower deck were omitted.

Consider the upper girder between Columns 2 and 3; the deflection of Column 2 produces a moment, which will be called M_1, at the point, A_2. As the structure is symmetrical about the point, A_3, the tangent to the deflection curve will remain horizontal at that point, or, in other words, the girder may be considered fixed at the point, A_3; and it follows that the moment in the girder at the point, A_3, is equal to one-half the moment in the girder at the point, A_2, giving a point of contraflexure, $\frac{2}{3}L$, from A_2.



Indicates Expansion Joint
or ◆ Indicates Rigid Connection

FIG. 2.

It will simplify matters to substitute for Girder $A_3\ A_2$, a girder which is rigidly connected to the column at A_2 and is freely supported on an expansion bearing at the other end; the length of girder must be such as to give the same moment, M_1, at Column 2. This length will be $\frac{3}{4}L$. It is somewhat more simple to work with a girder length equal to L, so it will be assumed that the girder has a length, L, and that its moment of inertia, I, is increased to $\frac{4}{3}I$.

Evidently, the moments in the column will be unchanged if, for the two girders connecting to the column at A_2, one girder is substituted having a moment of inertia equal to their sum; the moment of inertia of this one girder will be taken, therefore, as $\frac{4}{3}I + I = \frac{7}{3}I$, where I is the moment of inertia of the actual girder.

As the columns are battered from top to bottom on all faces, there is a considerable variation in moment of inertia throughout their

lengths. Any formulas that are applied for the solution of the desired moments, M_1 and M_2, must take into account this variation in moment of inertia of the column. Consider the column and attached girder as a beam on three supports having moments, M_0, M_1, and M_2, at the three supports and having a straight-line moment variation, as indicated in Fig. 3; the beam is assumed to be fixed at C.

Fig. 4 shows the beam as deflected by the moments.

Bearing in mind that the deflection of any point, as A_1, with respect to a tangent at A_2, is equal to the moment about A_1 of the area under the $\frac{M}{E\,I}$ curve, we can write the following equations:

$$\delta_1 = \int \frac{M\,x\,d\,x}{E\,I}.$$

But, $M = \frac{M_1\,x}{L}$, and I is constant for this span, therefore, we have

$$\delta_1 = \frac{M_1\,L^2}{3\,E\,I} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

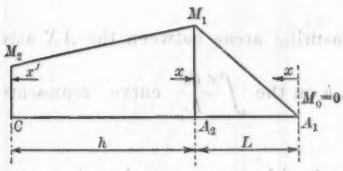


FIG. 3.

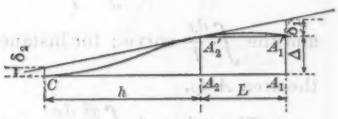


FIG. 4.

similarly, $$\delta_2 = \frac{M_2}{E}\int \frac{x'\,dx'}{I} + \frac{M_1}{E\,h}\int \frac{x'^2\,dx'}{I} - \frac{M^2}{E\,h}\int \frac{{x'_1}^2\,dx'}{I} \quad \ldots (2)$$

We have also the relation, $$\frac{\delta_1}{L} = \frac{\Delta - \delta_2}{h} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

We can eliminate the unknown quantities, δ_1 and δ_2 from Equations (1), (2), and (3), and have the following resulting equation:

$$\frac{M_1}{E}\left(\frac{L}{3\,E\,I} + \frac{1}{h^2}\int \frac{x'^2\,dx'}{I}\right) + \frac{M^2}{E}\left(\frac{1}{h}\int \frac{x'\,dx'}{I} - \frac{1}{h^2}\int \frac{x'^2\,dx'}{I}\right) = \frac{\Delta}{h} \quad \ldots (4)$$

A second equation involving only the unknown M_1 and M_2 can be written by considering the deflection of A_2 with respect to the tangent at C, as follows:

$$\frac{M_1}{E}\left(\frac{1}{h}\int \frac{x\,dx}{I} - \frac{1}{h^2}\int \frac{x^2\,dx}{I}\right) + \frac{M_2}{E}\left(\frac{1}{h^2}\int \frac{x^2\,dx}{I}\right) = \frac{\Delta}{h} \quad \ldots (5)$$

Numerical substitution in Equations (4) and (5) results readily in a solution of values of M_1 and M_2, the desired moments.

It might be of interest to explain the method used in ascertaining the values of the integrals, $\int \frac{x\,dx}{I}$, $\int \frac{x^2\,dx}{I}$, etc.

These integrals are plotted to scale on Fig. 5. The process of plotting is as follows:

1. Plot values of I, b represents the bottom of the girder—the deformation between A_2 and b was neglected, that is, I was assumed to be infinite.

2. Pl t values of $\frac{1}{I}$.

3. Plot values of $\int \frac{dx}{I}$; to locate a point of the $\int \frac{dx}{I}$ curve, measure the area between the $\frac{1}{I}$ curve, the line, $A\,C$, and the ordinate at the point; for instance, f on the $\int \frac{dx}{I}$ curve represents the area, $bcde$.

4. Plot values of $\int \frac{x\,dx}{I}$ by measuring areas between the AX axis and the $\int \frac{dx}{I}$ curve; for instance, h on the $\int \frac{x\,dx}{I}$ curve represents the area $Abfg$.

5. The value of $\int \frac{x^2\,dx}{I}$ was obtained by measuring the whole area between the $\int \frac{x\,dx}{I}$ curve and the AX axis.

The differential areas are shown on Fig. 5 and make clear, without further explanation, the correctness of these methods.

Stresses Due to Wind Loads.—In calculating these stresses, we have a 2-story framed bent to consider; the fact that the columns have a variable moment of inertia complicates the problem, and it is necessary to use formulas taking this into account. Further complication results from the fact that the two columns in the bent are unlike, with different elastic properties.

To solve the problem and ascertain the desired moments in the columns and cross-girders, the theorem of three moments is applied.

The bent under consideration is shown in Figs. 6 and 7.

Under the influence of the wind loads, H_1 and H_2, the frame will deflect in some such manner as that shown in Fig. 7. In this diagram

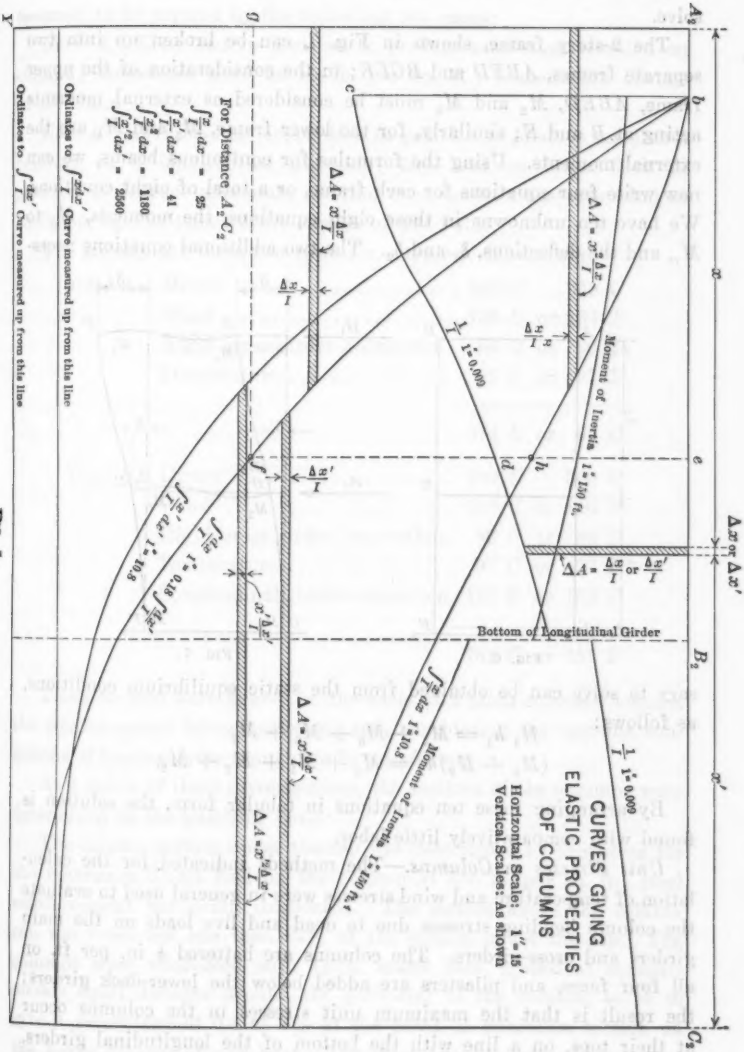

CURVES GIVING
ELASTIC PROPERTIES
OF COLUMNS
Horizontal Scale: 1" = 15'
Vertical Scales: As shown
Moment of Inertia 1" = 150 ft.
Bottom of Longitudinal Girder
For Distance A_2C_2
$\int \frac{x}{I} dx$ = 25
$\int \frac{x'}{I} dx'$ = 41
$\int \frac{x^2}{I} dx$ = 1180
$\int \frac{x'^2}{I} dx'$ = 2560
Ordinates to $\int \frac{x dx'}{I}$ Curve measured up from this line
Ordinates to $\int \frac{dx'}{I}$ Curve measured up from this line
$\Delta A = x \frac{\Delta x}{I}$
$\Delta A = x^2 \frac{\Delta x}{I}$
$\Delta A = x' \frac{\Delta x'}{I}$
$\Delta A = x'^2 \frac{\Delta x'}{I}$
$\Delta A = \frac{\Delta x}{I}$ or $\frac{\Delta x'}{I}$
$\frac{1}{I}$ 1" = 0.009
$\frac{1}{I}$ 1" = 0.008
$\frac{\Delta x}{I}$
$\frac{\Delta x'}{I}$
Δx or $\Delta x'$
A_2
B_2
C_2
a
b
c
d
e
g
h
x
x'
Y

FIG. 5.

also are shown the unknown moments, M_0 to M_7, for which we must solve.

The 2-story frame, shown in Fig. 7, can be broken up into two separate frames, *ABED* and *BCFE*; in the consideration of the upper frame, *ABED*, M_2 and M_3 must be considered as external moments acting at *B* and *E*; similarly, for the lower frame, M_5 and M_4 are the external moments. Using the formulas for continuous beams, we can now write four equations for each frame, or a total of eight equations. We have ten unknowns in these eight equations, the moments, M_0 to M_7, and the deflections, δ_1 and δ_2. The two additional equations neces-

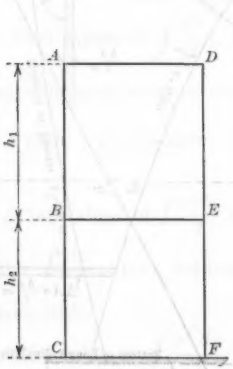


FIG. 6.

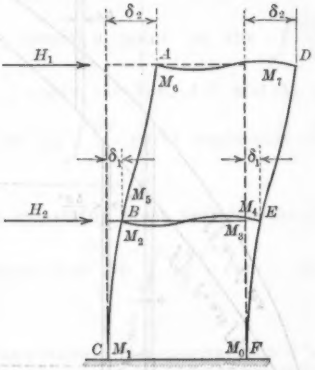


FIG. 7.

sary to solve can be obtained from the static equilibrium conditions, as follows:

$$H_1 h_1 = M_4 + M_5 + M_6 + M_7$$
$$(H_1 + H_2)h_2 = M_0 + M_1 + M_2 + M_3.$$

By arranging these ten equations in tabular form, the solution is found with comparatively little labor.

Unit Stresses in Columns.—The methods indicated for the calculation of temperature and wind stresses were in general used to evaluate the column bending stresses due to dead and live loads on the main girders and cross-girders. The columns are battered ⅛ in. per ft. on all four faces, and pilasters are added below the lower-deck girders; the result is that the maximum unit stresses in the columns occur at their tops, on a line with the bottom of the longitudinal girders. Various probable combinations of loading were considered; the maxi-

mum probable stresses that would ever occur in the columns were assumed to be covered by the following two cases:

Case A.—Live load in position to produce maximum reaction on column; transverse wind at 15 lb. per sq. ft.; temperature change of 40° Fahr.

Case B.—Live load on upper deck in position to produce maximum bending in a longitudinal direction; live load on lower deck in position to produce maximum reaction on column; transverse wind at 15 lb. per sq. ft.; temperature change of 40° Fahr.

The stresses for these two cases for a typical column were:

Case A:	Direct	355 *C*	355 *C*
	Wind	136 *C* or	136 *T*
	Rigid cross-girder connection.	86 *C* or	86 *T*
	Temperature	97 *C* or	97 *T*
		674 *C* or	36 *C*
Case B:	Direct	300 *C*	300 *C*
	Wind	116 *C* or	116 *T*
	Rigid cross-girder connection.	86 *C* or	86 *T*
	Temperature	97 *C* or	97 *T*
	Longitudinal girder connection.	182 *C* or	182 *T*
		781 *C* or	181 *T*

These stresses were figured on the basis of a plain concrete column, the reinforcement being neglected, and, considering the extreme conditions of loading, were thought to be satisfactory.

As a result of these investigations, the sections of the columns were determined on the following basis:

The column section under the cap was determined by providing for the maximum direct load on the column at 350 lb. per sq. in., steel reinforcement not being considered. The section above the column cap was made not less than 92% of the section under the cap. The columns were reinforced with 1¼-in. round bars, spaced at approximately equal distance around the perimeter of the column, so as to give a percentage of 0.8 of 1% at the point of minimum section on the heavy column of any bent. The steel for the light columns was then determined by using the same spacing as for the heavy column.

B.—Design of Arch.

Other designing problems of special interest are in connection with the arch at Santa Fé Street. This span has a clear opening of 130 ft., and consists of two arch ribs transversely spaced so that the lower roadway passes between them.

The two arch ribs support spandrel columns which, in turn, carry cross-girders of the usual type and the cantilevers of the upper deck. The lower deck is suspended from the arch ribs. Because of the very limited distance between the surface of the roadway on the lower deck and the clearance required above the railroad tracks below, the lower deck is composed of steel I-beams encased in, and carrying a slab of, concrete. These cross-beams rivet into continuous longitudinal steel beams which are supported at intervals of 17 ft. by hangers composed of two steel channels.

The horizontal thrusts of the arches are taken by the bottom chords of the steel eye-bars. This avoids the use of abutments large enough to resist the thrust—there being in fact insufficient room for such abutments—and affords several advantages because of the resulting ability to shorten the lower chords. Because of the short length of the hangers near the springing points of the arch, and the rigidity of the lower-deck girder, it was considered advisable to make the arches of the fixed type. One end of each arch rib is fixed by being built into the supporting pier, and the other end rests on a tandem rocker bent placed so as to keep the resultant thrust of the arch between the two bearings for every condition of loading. The springing points of the arch are relatively on the same grade as the lower roadway, so that the east springing point is 3.5 ft. higher than the west one. This condition causes some unusual longitudinal thrusts.

All the structural steel members of the span are encased in concrete, and the smaller ones are surrounded by light concrete walls giving dimensions for satisfactory architectural appearance.

The arches were designed under the usual elastic theory. The calculations were carried through for both unit vertical and unit horizontal loads at the various points, vertical loads in this instance meaning those perpendicular to, and horizontal loads meaning those parallel to, the arch axis. These latter loads include those due to thrust from the street-car loading, as well as the components of the dead and live loads caused by the grade.

The maximum stresses were calculated at each springing point, at the crown, and at seven intermediate points on each half of the ring—a total of seventeen different points. All calculations were carried through in appropriate tabular form, and are hardly of sufficient interest to warrant their inclusion in this paper. The influence lines for maximum moments at various points, due both to vertical and to horizontal loads, were plotted and are shown on Plate XVII.

Considerable experience in actual designs recommends highly the influence-line methods for a check on calculations. Errors seem to be invariably made evident by the failure of the various ordinates to any influence line to show a smooth curve. In the diagrams, it will also be noted that the apices of the various moment curves themselves lie on a smooth curve, a further check on the numerical work.

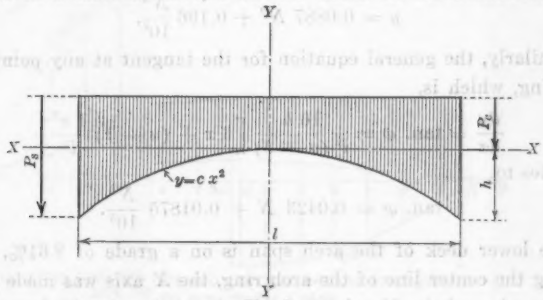


FIG. 8.

It was assumed that the intensity of loading on the arch ring, including the weight of the ring itself, would vary as the ordinates of a parabola, a condition of loading shown by Fig. 8; and the center line of the arch ring was located in the following manner:

Let P_c = equivalent uniform load at the crown;

P_s = equivalent uniform load at the springing;

$$u = \frac{P_s}{P_c};$$

h = rise of arch ring = 24 ft.;

l = span of arch ring = 134 ft.

Then, the equation of the funicular polygon results as follows:

$$y = \frac{4\,h}{l^2\,(u+5)}\left[6\,x^2 + (u-1)\,\frac{4\,x^4}{l^2}\right].$$

This equation was used as the equation for the center line of the arch ring, using dead load plus one-half live load in substituting

the numerical values of P_c and P_s. This substitution gave slightly different values of u for the two arch rings; and therefore an average value of $u = 1.34$ was used.

The equation was further simplified for use by dividing each half of the arch ring into sixteen equal horizontal sections, and numbering the division points successively from the crown to the springing.

Let N = number of any division point;
= 0 at crown;
= 16 at springing;

Then, $X = \frac{N\,l}{32}$.

Substituting the values of u, h, l, and x in the original equation, we have the following equation for the center line of the arch ring:

$$y = 0.0887\ N^2 + 0.196\ \frac{N^4}{10^4}.$$

Similarly, the general equation for the tangent at any point of the arch ring, which is,

$$\frac{dy}{dx} = \text{tan. } \phi = \frac{16\ h}{l^2\ (u + 5)} \left[3\ x + (u - 1)\ \frac{4\ x^3}{l^2}\right]$$

simplifies to,

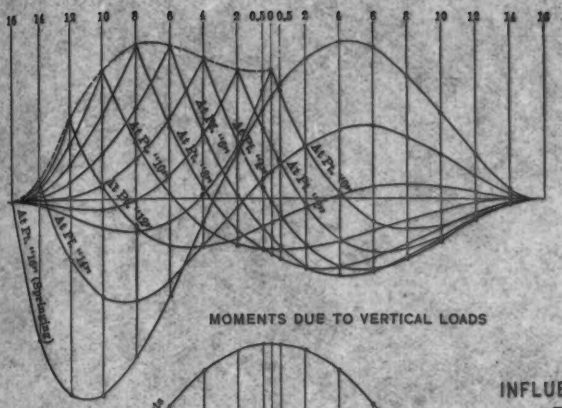
$$\text{tan. } \phi = 0.0423\ N + 0.01875\ \frac{N^3}{10^3}.$$

The lower deck of the arch span is on a grade of 2.61%, and in locating the center line of the arch ring, the X axis was made parallel to the grade and the Y axis perpendicular to it. As the hangers supporting the lower deck are vertical, it is evident that the dead loads are not applied symmetrically with respect to the crown of the arch ring. For purposes of calculation, it was assumed that the loads were applied symmetrically; this resulted in an error of less than 1%, which was considered negligible.

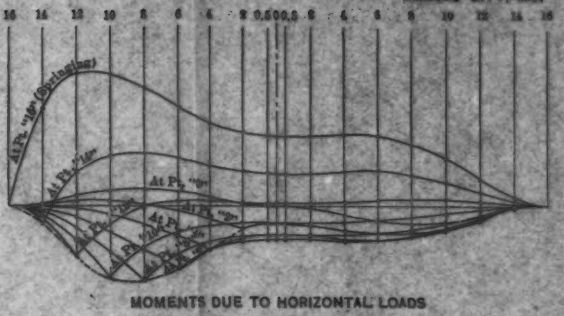
The use of the bottom chords of the eye-bars to take the thrust eliminates all temperature stresses in the arch rings, with the exception of those due to a possible variation in temperature between the arch ring and the bottom chord, which were considered negligible.

Toggle arrangements to shorten the lower chords were provided to accomplish the following:

1.—To take up the stretch in the bottom-chord eye-bars, due to dead load, the result being to leave the dead-load stresses in the arches the same as for fixed arches supported on rigid abutments.

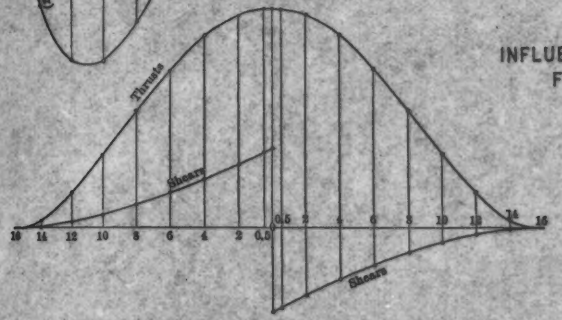


MOMENTS DUE TO VERTICAL LOADS



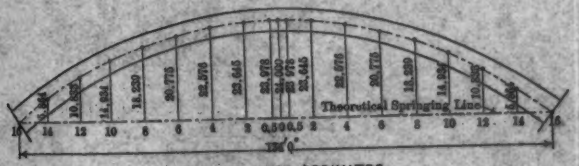
MOMENTS DUE TO HORIZONTAL LOADS

INFLUENCE LINE DIAGRAMS FOR ARCH SPAN



HORIZONTAL THRUSTS AND VERTICAL SHEARS AT CROWN DUE TO VERTICAL LOADS

HORIZONTAL THRUSTS AND VERTICAL SHEARS AT CROWN DUE TO HORIZONTAL LOADS



ARCH RIB WITH ORDINATES

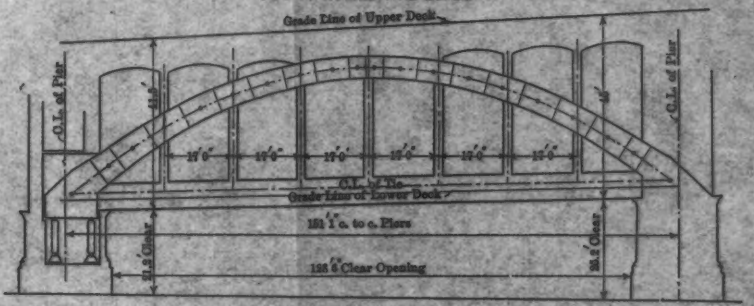


DIAGRAM OF GENERAL DIMENSIONS OF ARCH SHOWING ASSUMED DIVISIONS OF ARCH RIBS

2.—To shorten the bottom chords by amounts sufficient to eliminate the stresses in the arch rings due to their shortening under dead load.

3.—To shorten the bottom chords by the amounts which they would stretch, and the arch ribs would shorten, under full live load, over one-half the span.

As indicated by the influence lines for maximum moments at various points of the arch ring, the loaded length for a maximum moment is generally about one-half the span length; the result of shortening the bottom chord as indicated under Paragraph 3 is, therefore, to give practically the same maximum stresses in the arch ring as would result in a fixed arch supported on abutments in the ordinary way, with the further advantage that the stresses due to rib shortening are eliminated.

All these provisions required a total shortening of the bottom chords by the toggles of $1\frac{1}{16}$ in., which meant that the distance between the springing points, after the dead load was placed on the arches, was $\frac{3}{4}$ in. shorter than when the arches were constructed on the falsework.

4.—Specifications for Materials and Workmanship.

The specifications were prepared with a view to letting a unit-price contract after competitive bidding. Every effort was made to have them full and precise in description, both of materials and of workmanship, eliminating as far as possible the uncertain characterization of work "satisfactory to the Engineer" by expressing, wherever feasible, the views of the engineers in advance, and attempting to make perfectly clear to a prospective builder just what would be required of him. Though definite and explicit in the description of results desired, the specifications left open as far as possible the methods by which such results were to be secured, in order that the City might receive the full benefit of any contractor's skill and ingenuity in laying out and handling the work, and enjoy a portion of the economies thus effected.

Approximate quantities of various units of the structure were prepared and tabulated by the engineers. The City received bids on a unit basis of price per cubic yard for concrete in various divisions of the structure, price per pound for reinforcing material, for structural steel, etc., and the total bids were compared on the basis of the approximate quantities.

Concrete.—The specifications required the concrete of the structure to be made of 1 part cement, of detailed requirements, conforming in most respects to the recommendations of the American Society for Testing Materials; of 2 parts sand, "particles of hard, clean stone in graded sizes which shall pass a sieve having holes ¼ in. square, and not less than 50% be retained on a No. 30 sieve"; and 4 parts broken stone, "pieces of hard and durable rock entirely free from clay, loam, or any other material than sound, hard stone—of crusher-run sizes—with all material that will pass a ¼-in. sieve removed", to pass a 1½-in. ring.

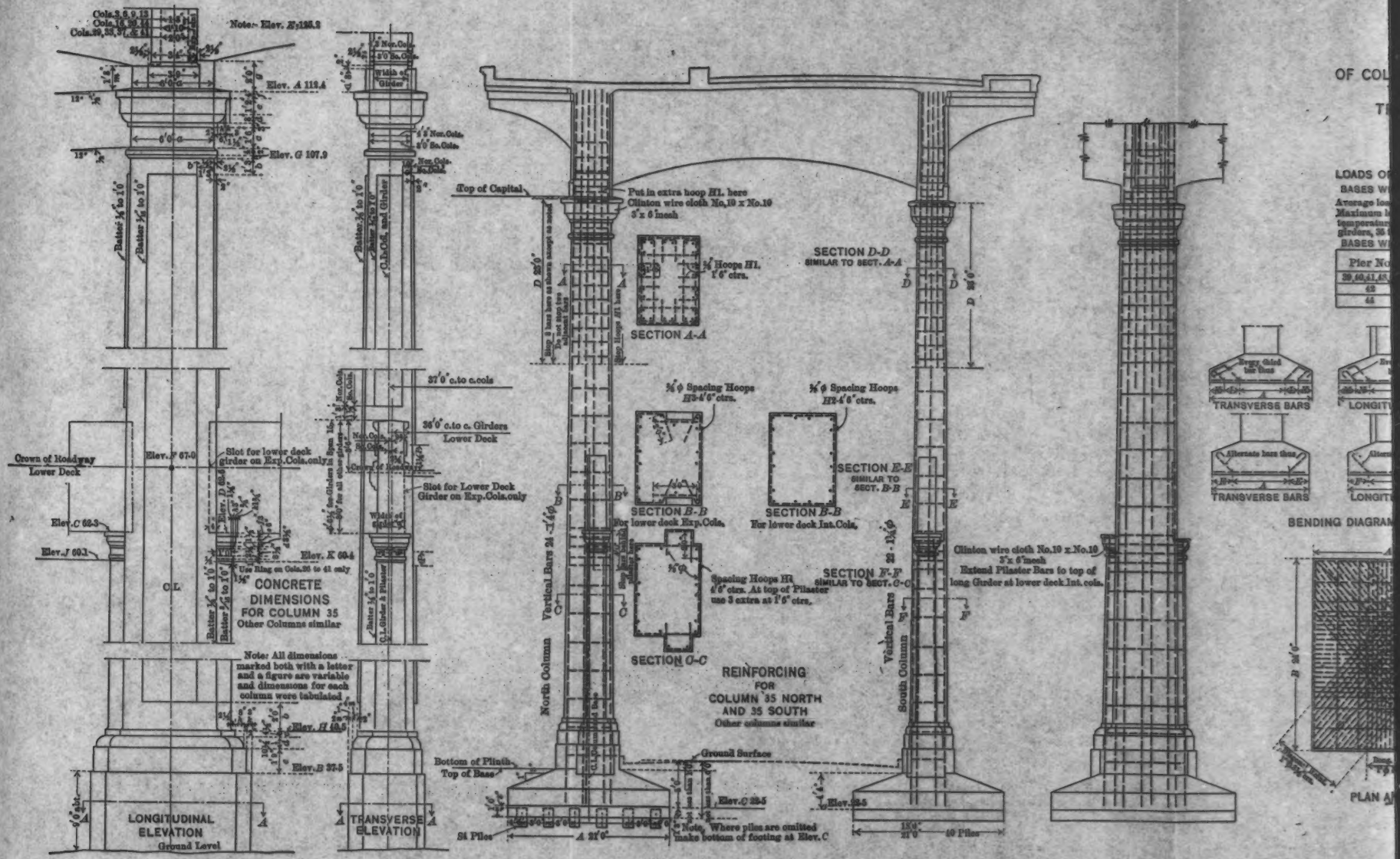
For the hand-rails, stair risers, and treads, a proportion of 1 part cement, 1½ parts sand, and 3 parts broken stone, to pass a ¾-in. ring, was required. The proportions are stated to be by volume, with the ingredients measured loose, one 380-lb. bbl. of cement being taken as 4 cu. ft. The upper ¾ in. of the sidewalk slab is of mortar of 1 part cement to 1½ parts sand placed before the supporting slab had set. The use of a mechanical batch mixer is specified, to be operated long enough after the last ingredient is deposited in it thoroughly to mix and incorporate all ingredients.

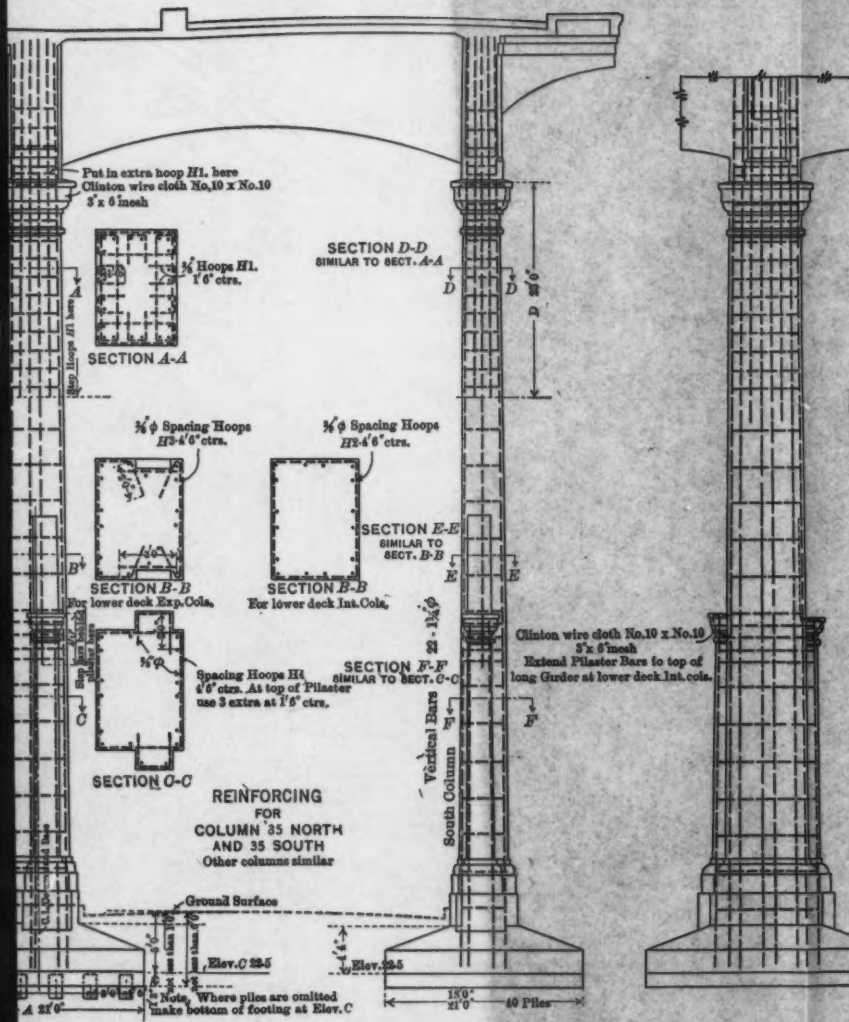
Reinforcement.—The reinforcement is of corrugated bars, "deformed bars having their corrugations at right angles to the length of the bar" having been required, rolled from open-hearth billets of medium steel having an ultimate strength of from 60 000 to 75 000 lb., and a minimum elastic limit of 35 000 lb., and affording an elongation of not less than 22% in 8 in.

The specifications provide that the "reinforcement in the finished structure shall conform accurately in size and position to the requirements of the plans. Bars shall be firmly bound and tied together by wire where they lap or cross, or be fastened by clips or other devices. Each piece must be held rigidly and positively in position, so that there will be no displacement during the depositing of concrete".

Concrete Piles.—Precast reinforced concrete piles were designed, and were detailed on the drawings, but the specifications left some option to the contractor as to the exact type to be used, excluding, however, "piles in which fresh or unset concrete is placed against the soil". The contractor finally elected to use Raymond piles, which were accepted by the engineers.

A description of this well-known concrete pile is hardly necessary, but it may be said briefly to consist of a sheet-steel shell, spirally


Note:- Elev. E=125.2
Elev. A 112.4
Elev. G 107.9
Batter 1/2" to 1'0"
Crown of Roadway
Lower Deck
Elev. F 67-0
Slot for lower deck girder on Exp.Cols.only
Elev. C 62-3
Elev. D 63-4
Elev. J 60.1
Elev. K 60.4
Use Ring on Cols.36 to 41 only
C.L.
CONCRETE DIMENSIONS FOR COLUMN 35
Other Columns similar
Note: All dimensions marked both with a letter and a figure are variable and dimensions for each column were tabulated
Elev. H 48.5
Elev. B 37.5
LONGITUDINAL ELEVATION
Ground Level
TRANSVERSE ELEVATION
27'0" c.to c.cols
36'0" c.to c. Girders
Lower Deck
Slot for Lower Deck Girder on Exp.Cols.only
Top of Capital
Put in extra hoop H1. here
Clinton wire cloth No.10 x No.10
5'x 6'mesh
SECTION A-A
SECTION D-D
SIMILAR TO SECT. A-A
SECTION B-B
For lower deck Exp.Cols.
SECTION B-B
For lower deck Int.Cols.
SECTION E-E
SIMILAR TO SECT. B-B
SECTION F-F
SIMILAR TO SECT. C-C
SECTION C-C
Spacing Hoops H3 6'0" ctrs. At top of Pilaster use 3 extra at 1'6" ctrs.
Vertical Bars 24
Vertical Bars 22
North Column
South Columns
REINFORCING FOR COLUMN 35 NORTH AND 35 SOUTH
Other columns similar
Clinton wire cloth No.10 x No.10
5'x 6'mesh
Extend Pilaster Bars to top of long Girder at lower deck Int. cols.
Bottom of Plinth
Top of Base
Ground Surface
Elev.C 23.5
Elev.23.5
84 Piles
A 22'0"
Note, Where piles are omitted make bottom of footing at Elev.C
15'0"
21'0"
40 Piles
OF COL
TR
LOADS O
BASES W
Average loa
Maximum l
temperatur
girders, 35
BASES W
Pier No
39, 40, 41, 42
43
44
TRANSVERSE BARS
LONGITU
TRANSVERSE BARS
LONGITU
BENDING DIAGRAM
PLAN A


Put in extra hoop H1. here
Clinton wire cloth No.10 x No.10
3'x 6'mesh
SECTION A-A
SECTION D-D
SIMILAR TO SECT. A-A
SECTION B-B
For lower deck Exp.Cols.
SECTION B-B
For lower deck Int.Cols.
SECTION E-E
SIMILAR TO SECT. B-B
SECTION C-C
SECTION F-F
SIMILAR TO SECT. C-C
REINFORCING
FOR
COLUMN 35 NORTH
AND 35 SOUTH
Other columns similar
Clinton wire cloth No.10 x No.10
3'x 6'mesh
Extend Pilaster Bars to top of
long Girder at lower deck Int.cols.
Vertical Bars
South Column
Ground Surface
Elev.C 22.5
Elev.22.5
Note, Where piles are omitted
make bottom of footing at Elev.C
180-21'0"
10 Piles

TYPICAL DETAILS OF COLUMNS, CAPS, AND BASES TWELFTH STREET TRAFFICWAY VIADUCT KANSAS CITY, MO.

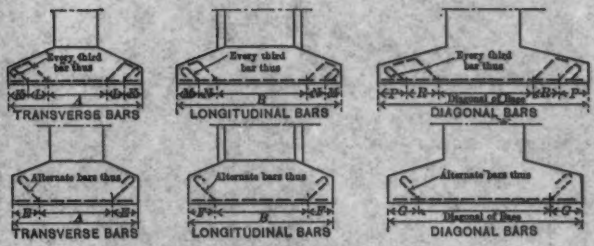
LOADS ON FOUNDATIONS

BASES WITH PILES

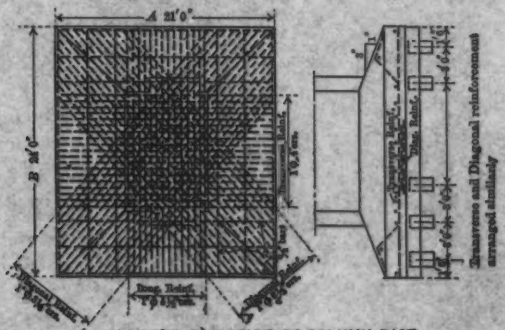
Average load on piles in each base, 25 tons per pile
Maximum load, considering effect of wind, temperature changes, and deflection of cross-girders, 35 tons per pile

BASES WITHOUT PILES:

Pier No.	Loads in Pounds per Square Foot	
	Av.	Max.
39,40,41,43,45	10 000	14 000
42	8 000	11 000
44	6 500	9 000



BENDING DIAGRAMS FOR BASE REINFORCEMENT



PLAN AND ELEVATION OF COLUMN BASE

reinforced, driven into the ground on a collapsible steel core. After reaching a satisfactorily resisting penetration, the core is withdrawn and the shell is filled with concrete. No reinforcing rods in the concrete were required, and as the concrete was not to be disturbed after placing, a mixture 1 part cement, 3 parts sand, and 5 parts broken stone was permitted.

Pavement.—The specifications for the creosoted wood block pavement for the lower roadway and on the street contain no unusual features. For steeper grades the following requirements are of interest:

"Creosoted block pavements, for roadways having a gradient exceeding 4%, are to be constructed in all parts of material and as specified for level roadways with the following exceptions: Creosoted laths about 1½ in. wide, ⅜ in. thick, and 4 ft. long, surfaced on one side, of uniform thickness, are to be furnished and placed on edge between every row of blocks continuously across the roadway, leaving a space about ⅜ in. wide and 2¾ in. deep between every row of blocks. The spaces between the blocks shall be filled with hot gravel which has passed through a sieve having holes ¼ in. square, and from which all sand and fine material which will pass a No. 20 sieve has been removed." * * * "Immediately after the gravel is in place, before it has cooled, paving pitch, as herein specified, shall be poured into it in such manner and quantity as entirely to fill all interstices."

This is practically the form of block pavement used in Vancouver, B. C., and elsewhere, on heavy grades. It affords a good grip for horses, and has proved very satisfactory.

Water-proofing.—The surfaces of both decks are covered with water-proofing in an effort to prevent the dripping of water on the roadways below. Under the ballast of the street-car area, this water-proofing consists of a two-ply burlap mat, laid down in water-proofing pitch, on which is placed a layer of asphaltic felt, and this, in turn, is covered with 1 in. of asphaltic mastic, and given a sand finish. Under the creosoted block pavement, one layer of burlap and one layer of asphaltic felt are laid in water-proofing pitch and given a sand finish. Indentations are formed in the concrete curbs some 6 in. below their tops, and the water-proofing mats are brought up, turned in, and finished in them. It happened that just before the contractor was to place his order for the burlap, the supply was entirely cut off owing to the European war conditions, so that felt was substituted, at some reduction of cost.

Lighting System.—The requirements for the lighting system were given as follows:

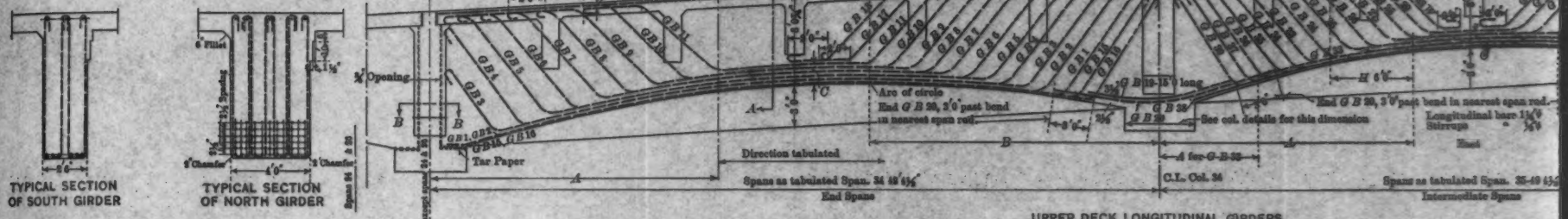
"The structure is to be illuminated by an electric lighting system. There shall be furnished and installed, all lamps, globes, conduits, wiring, and all other apparatus and appurtenances necessary for a complete series-Tungsten lighting system, taking current from the Kansas City Electric Light Company's 6.6-ampere, constant-current feeders at the east end of the structure. There will be two circuits. The circuit for lighting the upper roadway will have sixty 125-watt, 6.6-ampere, constant-current series-Tungsten lamps. Each lamp will be supported on a cast-iron standard, and will be surrounded by an opal glass globe, 14 in. in diameter. The circuit for lighting the lower roadway and the stairways will have fifty-six 75-watt, 6.6-ampere, constant-current, series-Tungsten lamps. Each lamp will be supported on a cast-iron standard or bracket, and will be surrounded by an opal glass globe, 12 in. in diameter. All wiring shall be drawn into Sherardized steel-pipe conduits of a satisfactory type, so as to be free from all flaws or mechanical injuries. All wires shall be No. 6, high-tension, lead-covered, Okonite, stranded, copper wire of the best quality. The conduit shall be encased in the concrete as the latter is placed, and shall be properly supported and arranged so that, later, the wire may be drawn into place. Suitable expansion joints shall be provided, and all connections shall be made in the conduits so that they will conform properly to the requirements of the structure. All joints between pipes and between pipes and boxes shall be made water-tight. The entire lighting system shall be constructed in a thoroughly workmanlike manner and to the satisfaction of the engineers."

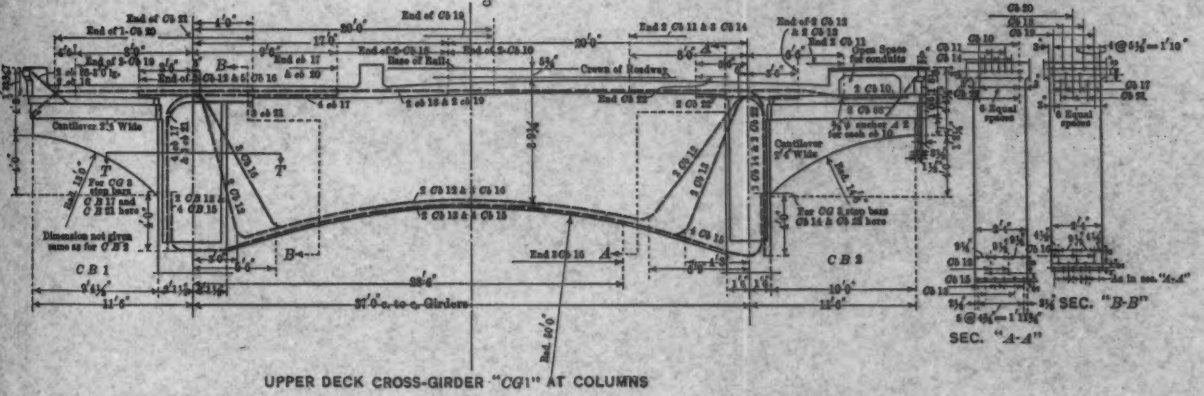
Other features of the specifications will be evident from the descriptions of methods of construction.

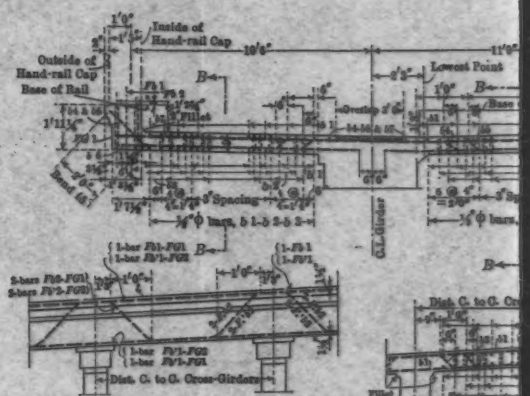
5.—Equipment for, and Methods of, Construction.

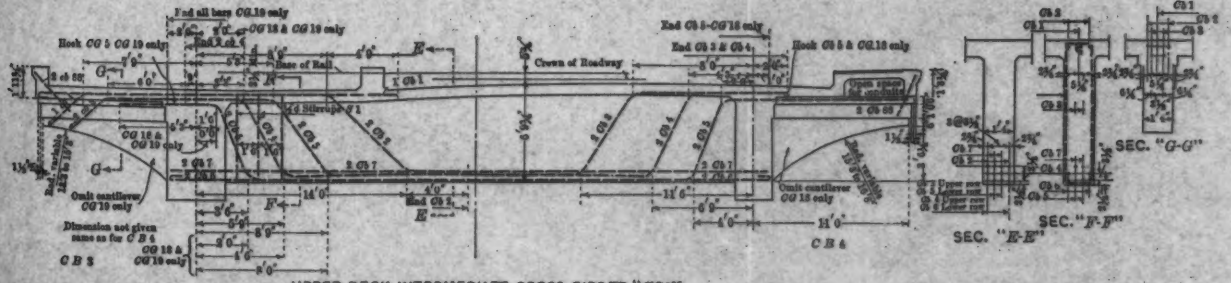
Yard Space.—The contractor's plant was at the west or low end of the viaduct, where some storage space, switch tracks, sidings, and facilities for handling materials, were more easily available than elsewhere on the right of way. The working space comprised practically all the ground from Liberty Street to Hickory Street, and from the north curb line of Eleventh Street to the alley south of Twelfth Street, an area of about 35 000 sq. ft.

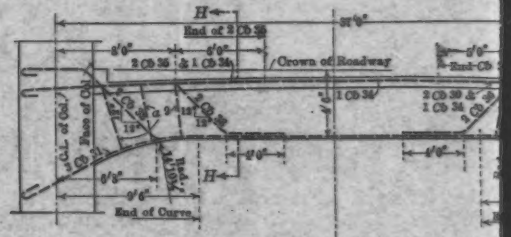
Three standard-gauge spur tracks were provided, one track paralleling Twelfth Street in the alley south of it—used for the delivery of timber—and two tracks crossing Twelfth Street at right angles—one for the delivery of sand, stone, and steel, and the other for cement.

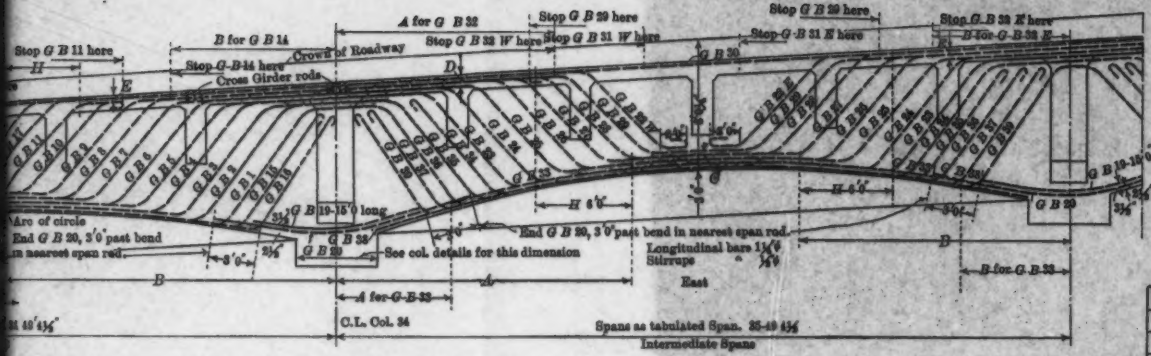

TYPICAL SECTION OF SOUTH GIRDER
TYPICAL SECTION OF NORTH GIRDER
Stop G B 11 here
Stop G B 12 here
Stop G B 13 here
Stop G B 14 here
Cross Girder rods
Crown of Roadway
A for G B 32
B for G B 14
Stop G B 32 here
Stop G B 31 here
Stop G B 30 here
Arc of circle
End G B 30, 2'0" past bend
End G B 30, 3'0" past bend in nearest span rod.
See col. details for this dimension
Longitudinal bars 1¼ ϕ
Stirrups ⅝ ϕ
Tar Paper
Direction tabulated
Spans as tabulated Span. 34 48'6½"
End Spans
C.L. Col.
Intermediate Spans
UPPER DECK LONGITUDINAL GIRDERS
Dimensions given for Spans 24 and 25


Cantilever 2'3 Wide
Cantilever 2'6 Wide
Dimension not given same as for CB 2
CB 1
CB 2
Crown of Roadway
Base of Rail
End of Cb 22
End of Cb 16
End 2 Cb 11 & 8 Cb 14
Open Space for conduits
6 Equal spaces
For CG 2 stop bars CB 11 and CB 21 here
For CG 3 stop bars Cb 14 & Cb 21 here
End 2 Cb 16
End Cb 16
Rad. 25'0"
25'0" c. to c. Girders
SEC. "A-A"
SEC. "B-B"
UPPER DECK CROSS-GIRDER "CG1" AT COLUMNS


Inside of Hand-rail Cap
Outside of Hand-rail Cap
Base of Rail
Lowest Point
Overlap 3'0"
5 Spacing
¾" ϕ bars, b 1-5 2-5 2-
1-bar FG1-FG2
1-bar FG1-FG2
Dist. C. to C. Cross-Girders
FASCIA GIRDERS FG-1 & FG-2
For Cross-Sections See Sect. A-A
¾" ϕ bars in FG1, ⅝" ϕ bars in FG2
SECTION

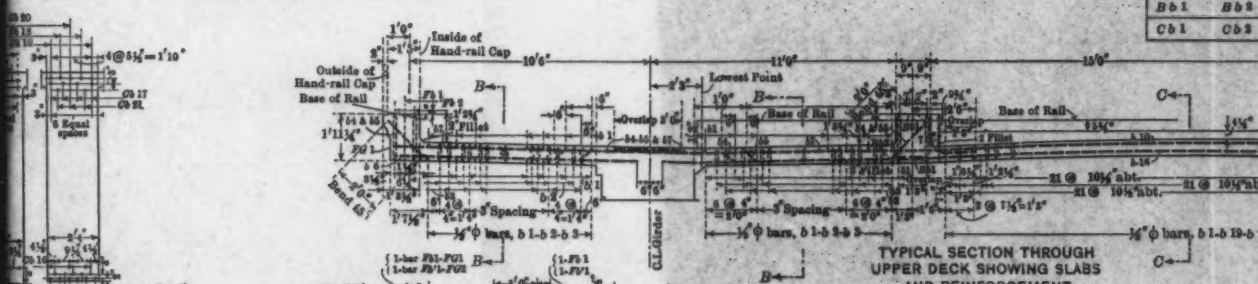

Hook CG 5 CG 19 only
End all bars CG 19 only
CG 18 & CG 19 only
Base of Rail
Crown of Roadway
End Cb 5-CG 18 only
Hook Cb 5 & CG 18 only
Unit cantilever CG 19 only
Dimension not given same as for CB 5
CB 3
CB 4
SEC. "E-E"
SEC. "F-F"
SEC. "G-G"
UPPER DECK INTERMEDIATE CROSS-GIRDER "CG2"


End of 2 Cb 36
2 Cb 36 & 1 Cb 36
Crown of Roadway
Base of Girder
End of Curve
LOWER DECK CROSS-GIRDER-CG 4
AT COLUMNS

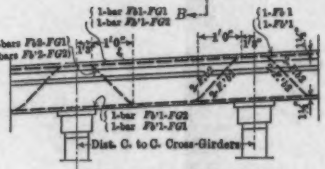


UPPER DECK LONGITUDINAL GIRDERS
Dimensions given for Spans 34 and 35

All bars ⅝"ϕ, exc	
b 10	b 16
b 29	b 30
F b 1	F b 2
F b'1	F b'2
B b 1	B b 2
C b 1	C b 2

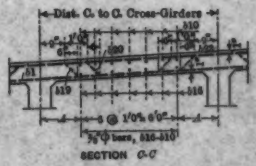


TYPICAL SECTION THROUGH
UPPER DECK SHOWING SLABS
AND REINFORCEMENT

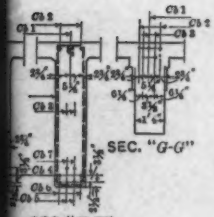


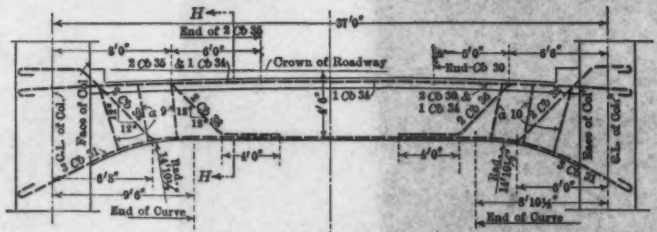
FASCIA GIRDERS *FG*-1 & *FG*-2
For Cross-Sections, See Sect. *A-A*
⅝"ϕ bars in *FG*1, ⅞"ϕ bars in *FG*2

SECTION *B-B*



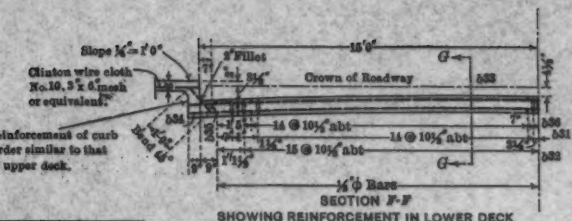
SECTION *C-C*





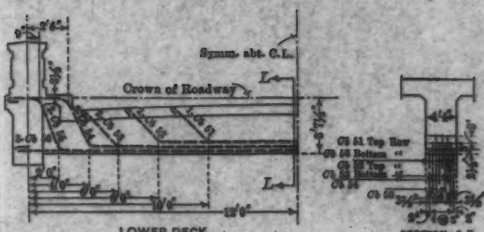
LOWER DECK CROSS-GIRDER-*C G 4*
AT COLUMNS

SECTION *H-H*

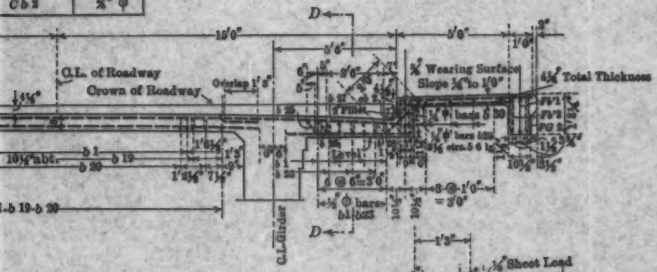


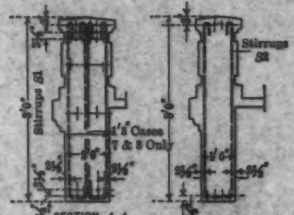
SECTION F-F
SHOWING REINFORCEMENT IN LOWER DECK
TYPICAL DETAILS OF
LOWER DECK FLOOR-SLABS

ϕ, except as follows,	
b 16	⅝" ϕ
b 20	¼" ϕ
F b 2	⅝" ϕ
F b'2	⅝" ϕ
B b 2	⅝" ϕ
C b 2	⅝" ϕ



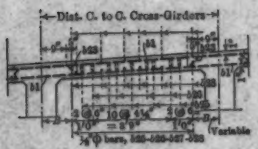
LOWER DECK
INTERMEDIATE CROSS-GIRDER - CG5



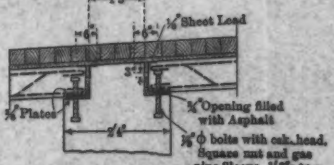


TYPICAL SECTION
OF NORTH GIRDER

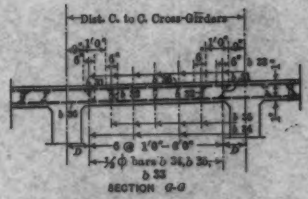
TYPICAL SECTION
OF SOUTH GIRDER



SECTION D-D

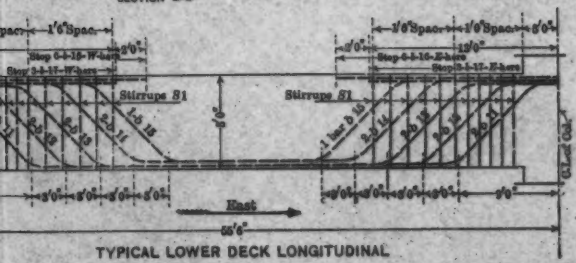


DETAIL OF EXPANSION JOINT
FOR ROADWAY SLABS



SECTION G-G

TYPICAL DETAILS OF
SLABS, GIRDERS AND
LONGITUDINAL GIRDERS
FOR BOTH DECKS
TWELFTH STREET
TRAFFICWAY VIADUCT,
KANSAS CITY, MO.



TYPICAL LOWER DECK LONGITUDINAL
GIRDER INTERMEDIATE SPAN

Concrete Plant.—The mixing plant, placed on the line of the viaduct, consisted of elevated bins for sand and broken stone arranged to discharge into measuring hoppers, below which a 33-cu. ft. Chicago-cube mixer was installed. The mixer dumped the concrete into hopper cars operating on a 30-in. gauge track passing underneath. Stone and sand were delivered in bottom-dump cars and unloaded into a receiving pit alongside the bunkers. A 22-in. Jeffery, belt-bucket elevator, of a total height of 70 ft., raised the stone and the sand from the pit to the top of the bins, where a baffle-board directed each to a proper bin. One elevator thus handled both sand and stone, unloading first one and then the other. The cement was hauled up an inclined track from the cement storage shed on a car, self-dumping at the top, to a platform under the sand bin, where it was filled into a sheet-iron tube with a bottom gate, and so discharged into the mixer. The tube was of just the proper dimensions to hold six sacks of cement, the normal charge. The limited room for tracks made necessary considerable shifting of the cars, as only the first car of a train could be placed over the pit. The operations of unloading and mixing were not always simultaneous, but when all operations were going forward, there were required, severally, two men for unloading and handling the elevator, three men for unloading cement from cars, one in the storage shed, two opening sacks and feeding to the mixer, and one man attending to charging and operating the mixer. Thus, the services of nine men were needed for the unloading and mixing of concrete ingredients.

During severe weather, the stone, sand, and water for the concrete were heated by passing steam from a 60-h.p. boiler into the bottoms of the sand and stone bins, and into the water, thoroughly and satisfactorily heating all the materials.

Equipment for Steel and Timber Reinforcing.—The steel-storage yard was adjacent to one of the switch tracks, and was equipped with a working platform and a power bar-bender of ingenious design. This bender was devised on the job, and included a motor connected by a worm gear to a vertical shaft which had at its upper end a clutch with a jaw to engage the bars to be bent, thus bending the bars in a horizontal plane on a protractor plate to any desired angle. The lumber-storage yard was beside the third siding. From this lumber yard, the material was passed into a wood-saw shop, thence into the carpenter shops, and then forward to the structure. The wood shop was equipped

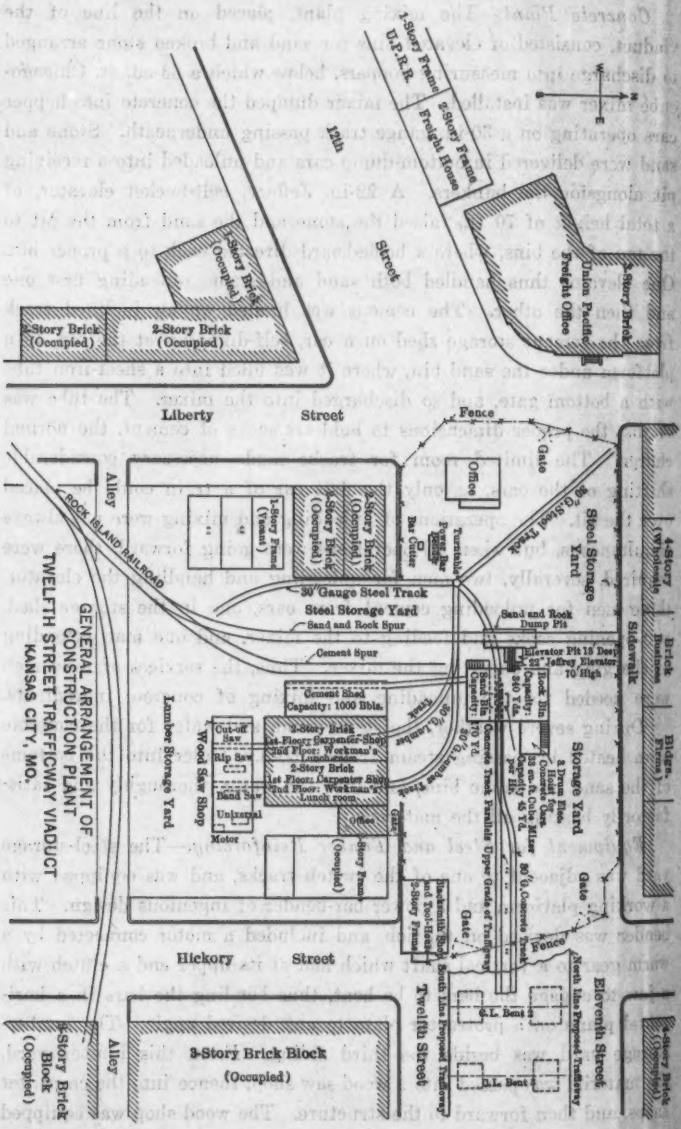
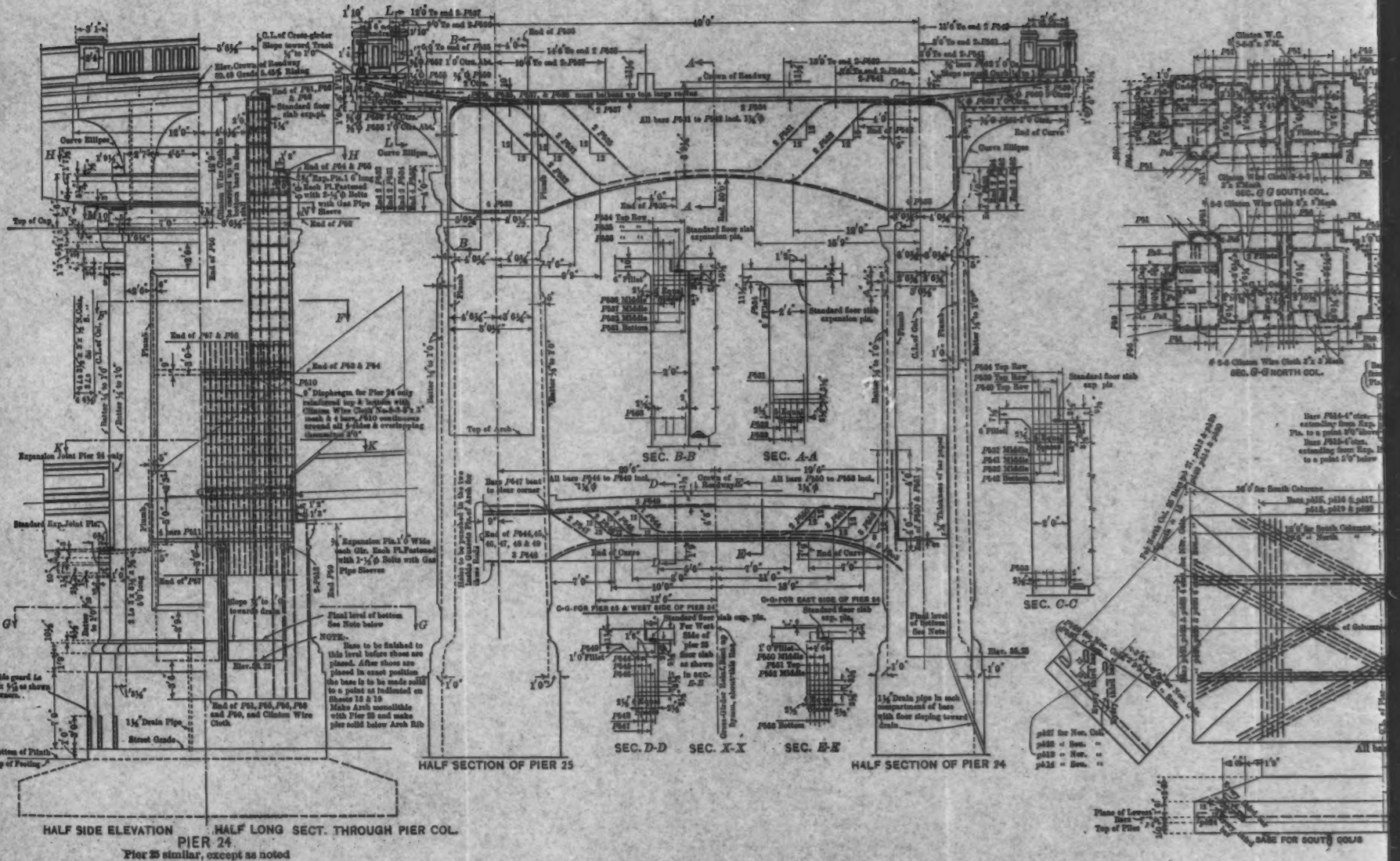

GENERAL ARRANGEMENT OF CONSTRUCTION PLANT
TWELFTH STREET TRAFFICWAY VIADUCT
KANSAS CITY, MO.
Liberty Street
Hickory Street
Twelfth Street
Eleventh Street
Alley
ROCK ISLAND RAILROAD
1-Story Frame U.P.R.R.
2-Story Frame Freight House
Union Pacific Freight Office
2-Story Brick
2-Story Brick (Occupied)
1-Story Frame (Vacant)
Office
Fence
Gate
Steel Storage Yard
Sand and Rock Dump Pit
Elevator Pit 15 Deep
25' Jeffrey Elevator 70' High
Cement Spur
Cement Shed Capacity: 1000 Bbls.
Lumber Storage Yard
Wood Saw Shop
Cut-off Saw
Rip Saw
Band Saw
Universal
Motor
2-Story Brick 1st Floor: Carpenter Shop 2nd Floor: Workman's Lunch room
Concrete Track Parallels Upper Grade of Trafficway
Blacksmith Shop and Tool-Room
3-Story Brick Block (Occupied)
4-Story (Wholesale)
Brick Business Bldgs. Fronts
Sidewalk
N.L. Bent
S.L. Bent
North line Proposed Trafficway
South line Proposed Trafficway

Fig. 9.



PIER 24

Pier 25 similar, except as noted

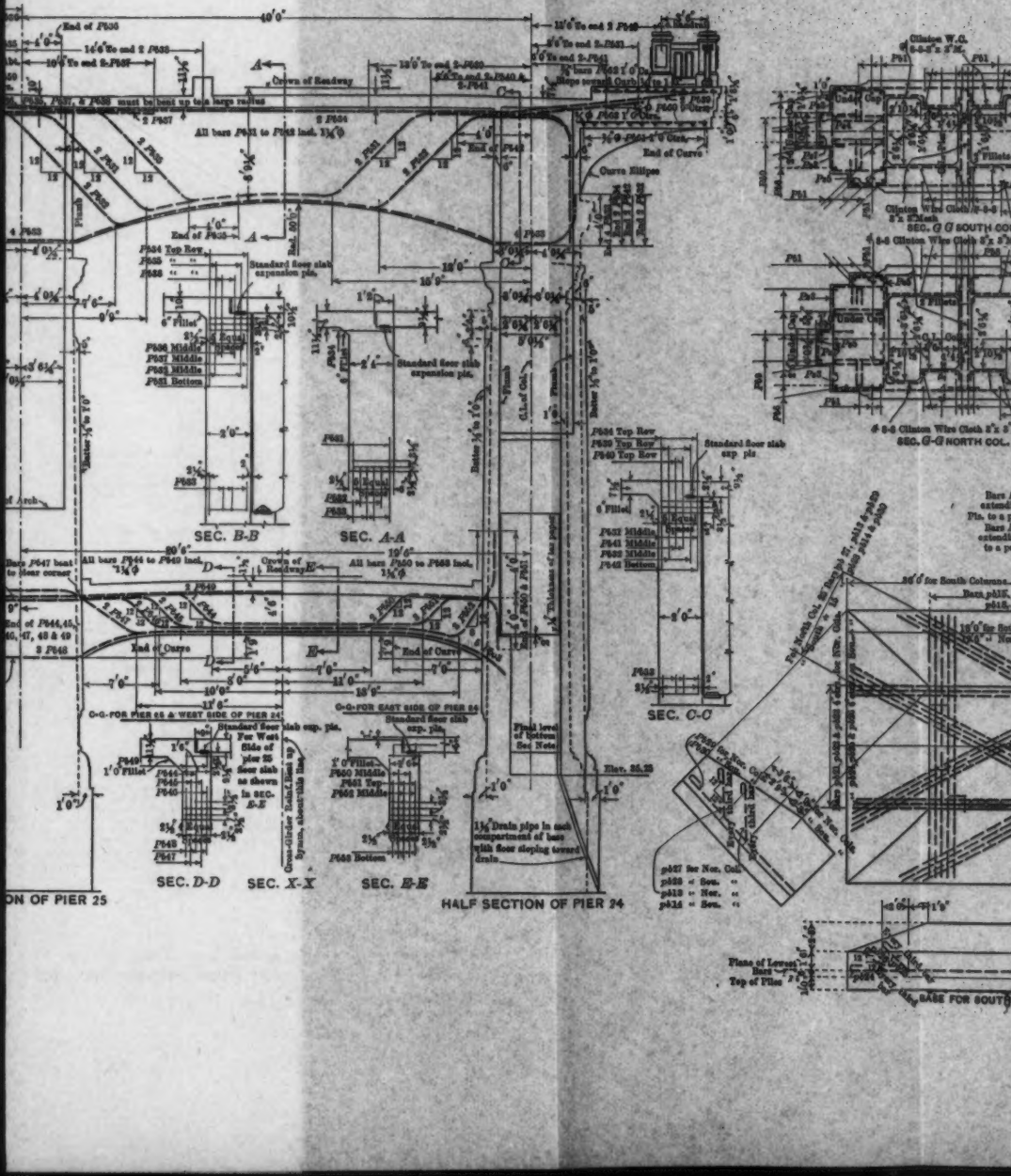
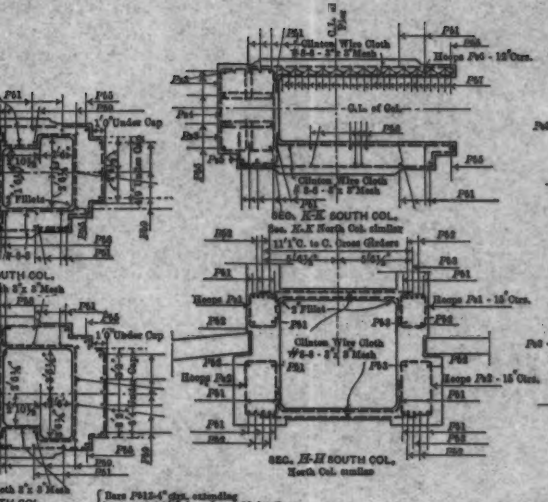
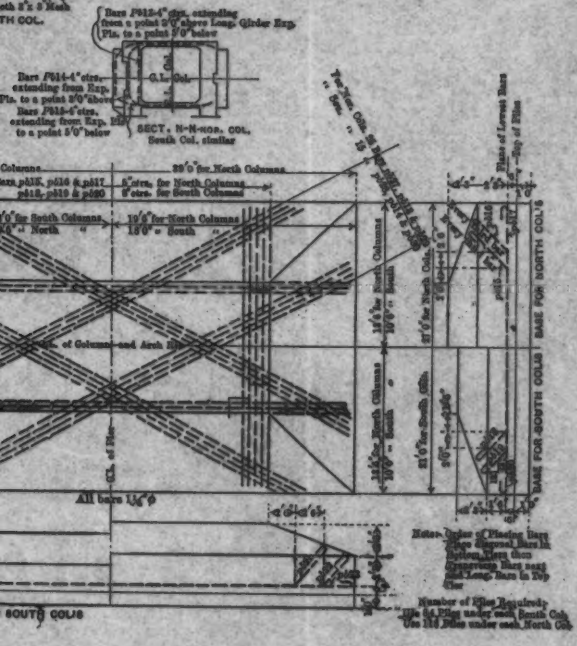

SEC. B-B
SEC. A-A
SEC. C-C
SEC. D-D
SEC. X-X
SEC. E-E
HALF SECTION OF PIER 24
SEC. G-G SOUTH COL.
SEC. G-G NORTH COL.
ON OF PIER 25
BASE FOR SOUTH

PLATE XX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXX, No. 1357.
HOWARD ON
TWELFTH ST. TRAFFICWAY VIADUCT,
KANSAS CITY, MO.





Mark	Size	Mark	Size
P# 1	1" ϕ	P# 13	⅞" ϕ
" 2	"	P# 14 & 15	¾" ϕ
" 3	"		
" 4	"		
" 5	"		
" 6	"		
" 7	⅞" ϕ		
" 8	1" ϕ		
" 9	"		
Hooping P#-1 to P#-3	⅝" ϕ		
P#-10, 11, & 12	¾" ϕ		

REINFORCEMENT IN COLUMN SHAFTS OF PIER 24

Reinforcement in Col. Shafts of Pier 25 Same as in Col. Shafts of Pier 24 except omit P#7, P#8, P#9 P#10 & P#11 Carry the other bars to 2'6" below top of Arch Ring except the bars in the circumference, which bars are to be carried to a point 5'0" below crown of lower deck roadway.

DETAILS OF PIERS OF ARCH SPAN AT SANTA FE STREET TWELFTH STREET TRAFFICWAY VIADUCT KANSAS CITY, MO.

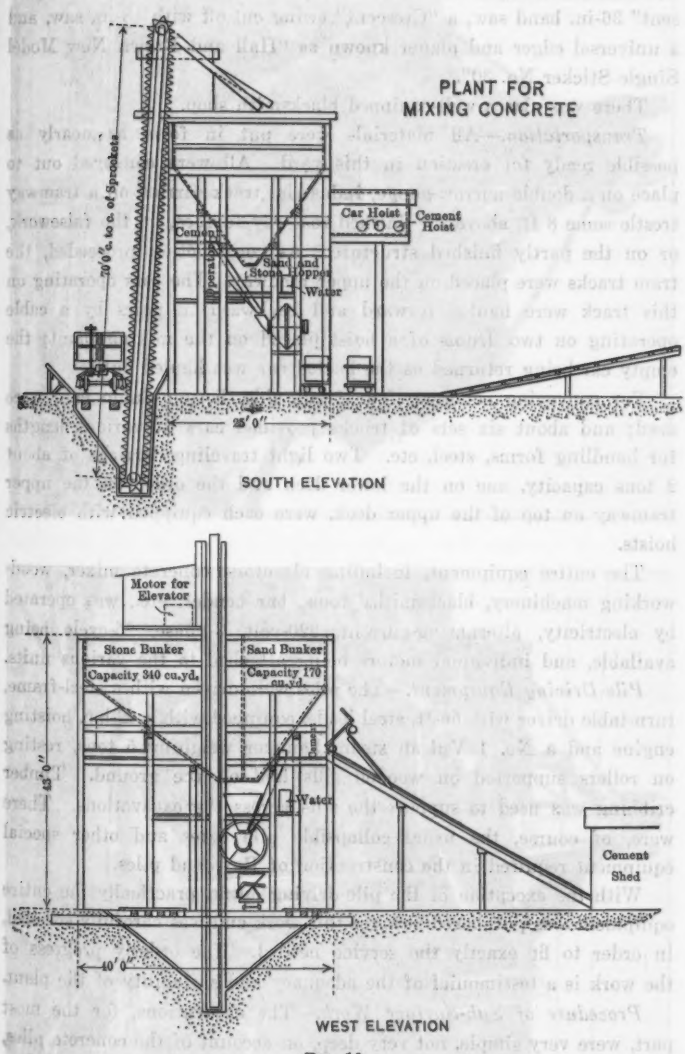

PLANT FOR
MIXING CONCRETE
70' 6" c. to c. of Sprockets
Car Hoist
Cement Hoist
Cement
Operating
Sand and Stone Hopper
Water
25' 0"
SOUTH ELEVATION
Motor for Elevator
Stone Bunker
Capacity 340 cu.yd.
Sand Bunker
Capacity 170 cu.yd.
Water
Cement Shed
43' 0"
40' 0"
WEST ELEVATION

FIG. 10.

with a circular rip-saw, a "Hall and Brown rip-saw No. 131", a "Crescent" 36-in. band saw, a "Crescent" swing cut-off with 18-in. saw, and a universal edger and planer known as "Hall and Brown New Model Single Sticker No. 30".

There was also a well-equipped blacksmith shop.

Transportation.—All materials were put in form as nearly as possible ready for erection in this yard. All were conveyed out to place on a double narrow-gauge, industrial track carried on a tramway trestle some 8 ft. above the finished roadway surface, on the falsework, or on the partly finished structure. As construction proceeded, the tram tracks were placed on the upper roadway. The cars operating on this track were hauled forward and backward in pairs by a cable operating on two drums of a hoist placed on the mixing plant; the empty car being returned as the loaded car was hauled out.

For conveying concrete, four 2-yd., side, bottom-dump care were used; and about six sets of trucks provided cars of various lengths for handling forms, steel, etc. Two light traveling derricks of about 2 tons capacity, one on the lower deck and the other on the upper tramway on top of the upper deck, were each equipped with electric hoists.

The entire equipment, including elevators, concrete mixer, woodworking machinery, blacksmiths' tools, bar bender, etc., was operated by electricity, alternating-current, 220-volt, 3-phase, 25-cycle being available, and individual motors being attached to the various units.

Pile-Driving Equipment.—The piles were driven with a steel-frame, turn-table driver with 56-ft. steel leads, equipped with a 42-h.p. hoisting engine and a No. 1 Vulcan steam hammer weighing 5 tons, resting on rollers supported on wooden sills laid on the ground. Timber cribbing was used to support the sills across the excavations. There were, of course, the usual collapsible pile cores and other special equipment required in the construction of Raymond piles.

With the exception of the pile-driving outfit, practically the entire equipment was purchased new for this work and was carefully selected, in order to fit exactly the service needed. The orderly progress of the work is a testimonial of the adequacy and suitability of the plant.

Procedure of Sub-Surface Work.—The excavations, for the most part, were very simple, not very deep, on account of the concrete piles, and in a sandy loam, easily dug, and standing without much shoring

FIG. 11.—BACK VIEW OF MATERIAL BINS AND MIXING PLANT, SHOWING PIT AND BELT BUCKET ELEVATOR.

FIG. 12.—FRONT VIEW OF MATERIAL BINS AND MIXING PLANT, SHOWING MIXER, CONCRETE CAR, HOIST, ETC.

or sheeting. Near the east end of the viaduct there was considerable old, filled material; and rock was encountered at various elevations. The east abutment rests in part on clay and shale with front toe-walls, 5 ft. thick, carried to rock at about 10 ft. lower elevation. The work was simple, open excavation.

After excavations were made, the Raymond concrete piles were driven, in the usual manner, to the refusal the apparatus would stand. The engineers required that all the steel shells for a group of piles be driven and then the concrete for all be placed at one time. This was to prevent any possible damage to the unset concrete of one pile by the driving of another near it. No particular difficulty resulted, except that the shells would sometimes partly fill with water or with sand and silt, which had to be pumped out before depositing the concrete. The shells stood open without collapse, except where water-bearing, so-called "quicksand" was encountered, and there it was found necessary to fill each shell as driven. In the main, the concrete for the piles was mixed in the general mixing plant, but, in some instances, for convenience, small quantities were mixed by hand.

Forms and Falsework.—The forms and falsework merit special attention, for their construction, erection, and removal formed the principal items of labor cost of the whole structure, and the efficiency with which the work was carried forward was dependent in no small degree on the thoroughness and care the contractor used in their design and construction.

Plans and sketches were prepared dimensioning all special parts of all forms, and, as far as possible, controlling portions were constructed in the carpenter shops and taken out entire for erection; only the principal portion of the sheathing was put on in place in the usual manner. Though the structure has an effect of duplication, there is, in reality, very considerable variation of dimensions within small limits, so that there were few parts of forms removable and replaceable without alteration. For instance, there are no two columns of the entire structure alike.

The forms for the columns were built to dimensions in the carpenter shops in three principal parts; a ready-to-erect form for plinth and for capital, and partly finished forms for the shafts, comprising four side frames of correct height and batter, from 8 to 12 in. wide at each corner, or rather extending back to the raised panel of

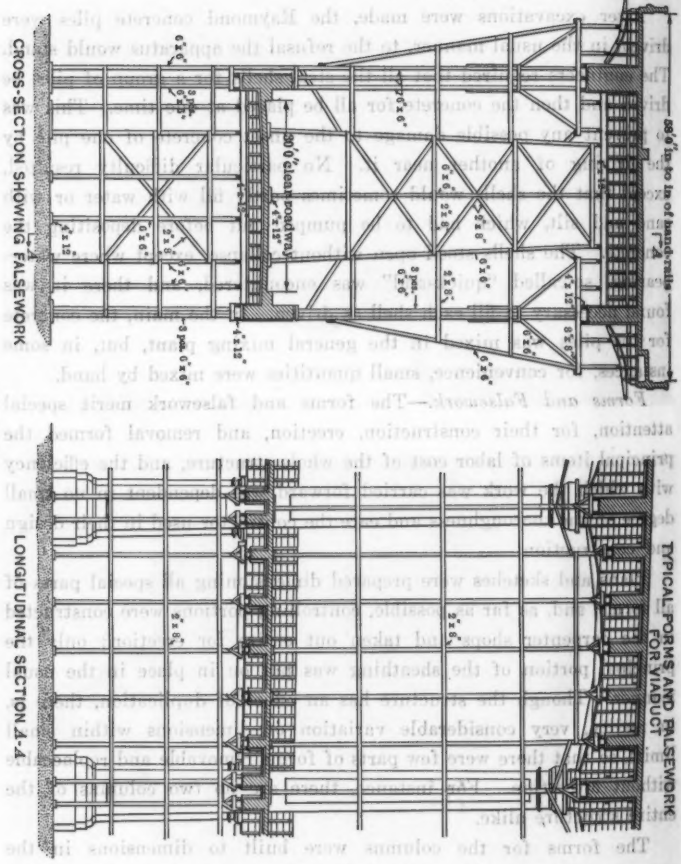
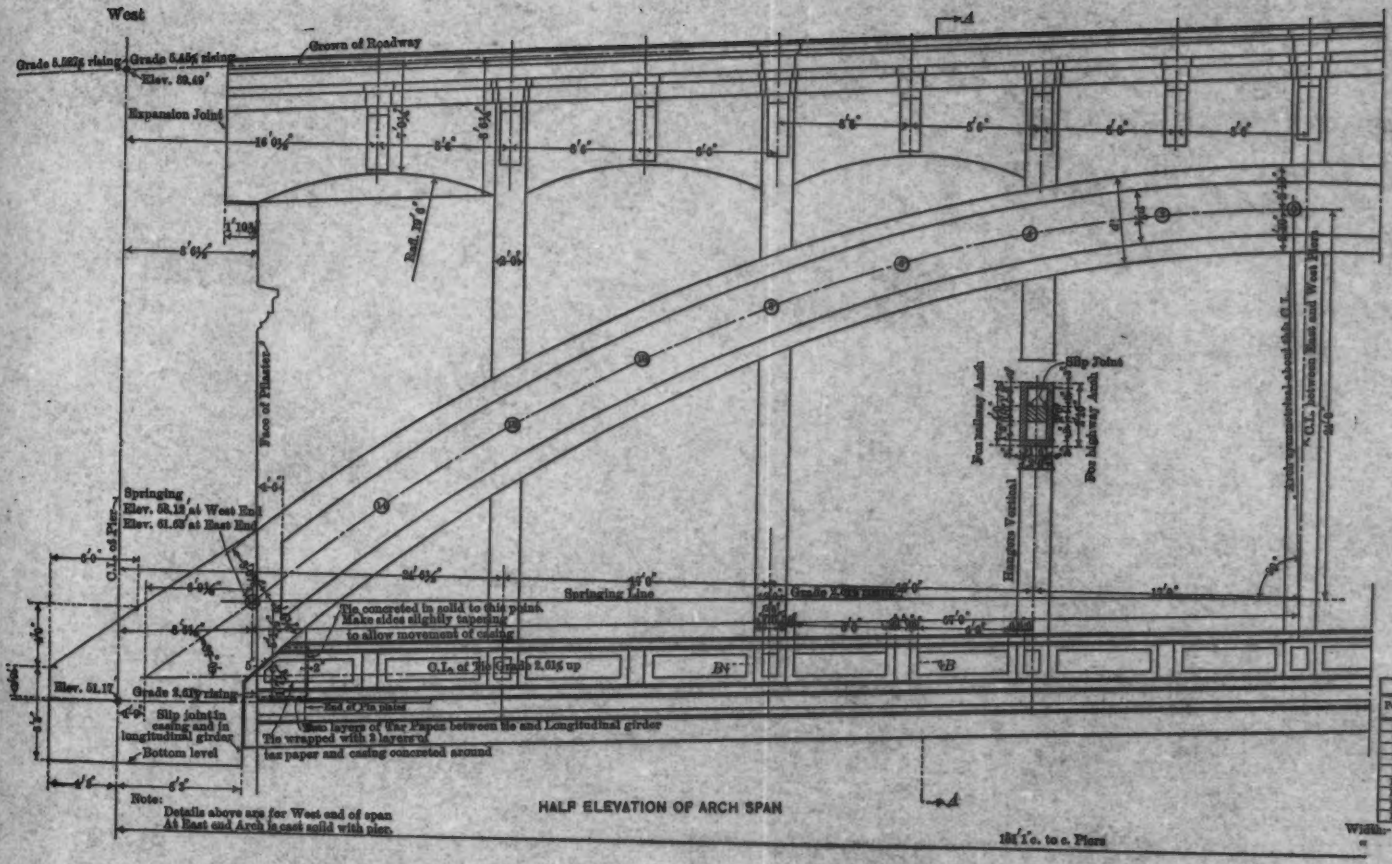
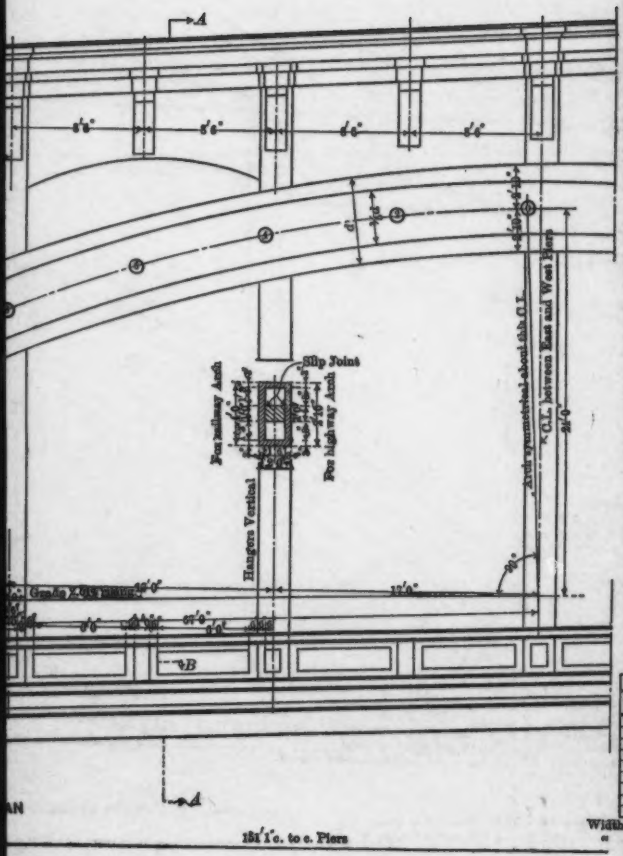

CROSS-SECTION SHOWING FALSEWORK
LONGITUDINAL SECTION A-A
TYPICAL FORMS AND FALSEWORK FOR VIADUCT

FIG. 13.



Point	Coordinates		Depth
	X in ft.	Y in ft.	in ft.
0	0	0	4.67
1	4.16	0.38	4.68
2	8.31	1.49	4.69
3	12.41	3.32	4.71
4	16.33	5.78	4.69
5	19.90	8.67	4.66
6	23.00	11.87	4.62
7	25.52	15.31	4.55
8	27.36	18.87	4.47
9	28.46	22.43	4.38
10	28.80	25.89	4.28
11	28.39	29.13	4.19
12	27.26	32.03	4.11
13	25.46	34.49	4.05
14	23.05	36.41	4.01
15	20.09	37.71	3.99
16	16.66	38.32	3.98
17	12.85	38.19	3.98
18	8.75	37.32	3.98
19	4.46	35.72	3.98
20	0	33.49	3.98



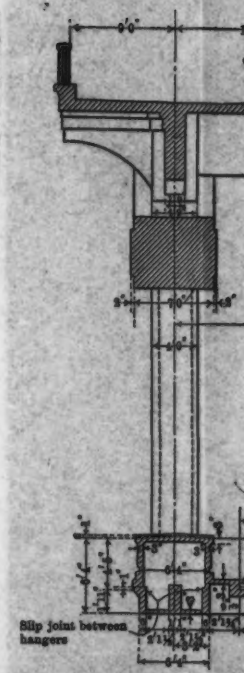
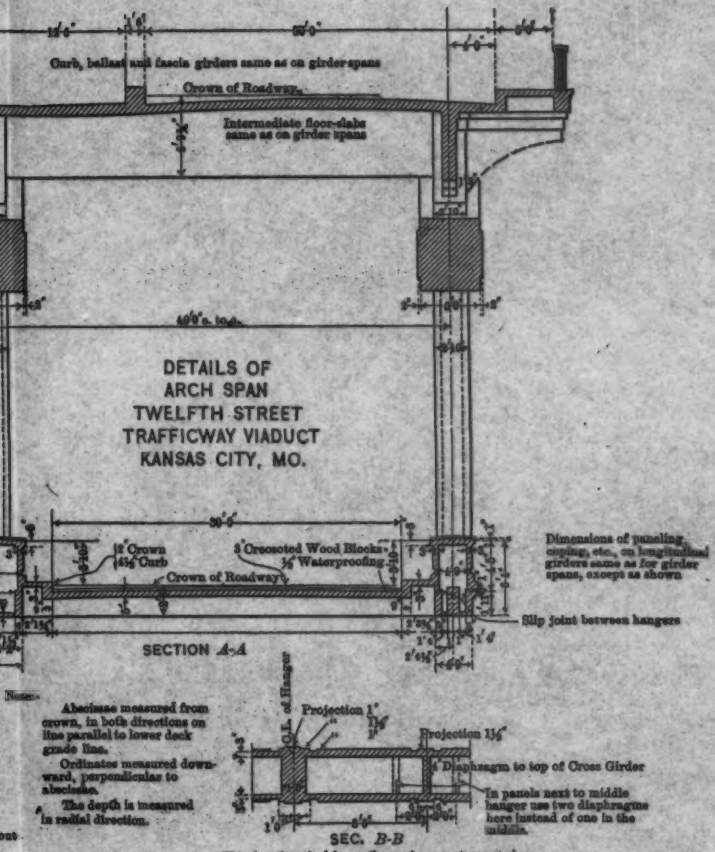


TABLE FOR ARCH RIB			
Point	Coordinates		Depth in ft.
	X in ft.	Y in ft.	
0	0	0	4.67
1	4.66	0.66	4.69
2	14.16	1.98	4.76
3	23.66	3.30	4.87
4	33.16	4.62	5.02
5	42.66	5.94	5.21
6	47.41	6.60	5.32
7	52.16	7.26	5.43
8	56.92	7.92	5.55
9	61.66	8.57	5.67
10	66.41	9.23	5.78
11	67.00	24.00	7.07

Note:-

Width:- Highway Arch, 5'0" throughout
" Railway " 7'9" "

PLATE XXI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXX, No. 1357.
HOWARD ON
TWELFTH ST. TRAFFICWAY VIADUCT,
KANSAS CITY, MO.



Notes.

Abscissæ measured from crown, in both directions on line parallel to lower deck grade line.

Ordinates measured downward, perpendicular to abscissæ.

The depth is measured in radial direction.

This detail typical for entire arch, except as noted

the column. The sheathing over the panels was prepared and placed after the side frames were adjusted to position. Yokes surrounding the column, of the usual four timbers bolted at the corners, and tied across with a central bolt surrounded by a tin tube, were placed at intervals and given bearing by wooden wedges.

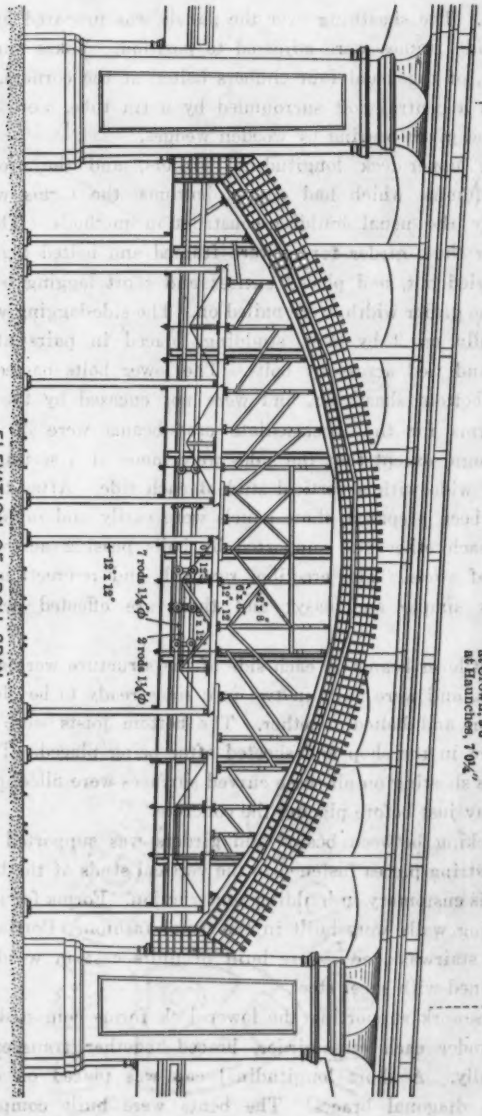
For the upper-deck longitudinal girders, and the cross-girders between columns, which had curved bottoms, the forms were built in place by the usual building-construction methods. The curved joists under these girder forms were framed and bolted together full length, carried out, and placed entire; and short lagging boards, the length of the girder width, were nailed on. The side-lagging was placed longitudinally on 2 by 6-in. studding, placed in pairs at suitable intervals, and tied across by bolts. The lower bolts passed through below the bottom sheathing, and were not encased by the concrete.

The forms for the intermediate cross-beams were of the same general design, except that the sides were made of a series of panels about 2 ft. wide with a vertical stud at each side. After the bottom forms had been prepared, these panels were easily and rapidly set up alongside each other and supported by bolts passing across between the pairs of studs. The erection, removal, and re-erection of these panels was simple and easy; and their use effected considerable economies.

The cantilever beams on each side of the structure were also duplicated forms, and were built in two unit-sides ready to be placed on a bottom form and bolted together. The bottom joists were sawed to correct form in the shop and sheeted after being placed. The cracks between the sheeting on all these curved surfaces were filled, or pointed up, with clay just before placing the concrete.

The decking between beams and girders was supported on joists resting on string pieces fastened to the vertical studs of the beams and girders, as is customary in building construction. Forms for abutments and retaining walls were built in the usual fashion. Portions of the hand-rails, stairways, etc., were built of units cast in wooden forms generally lined with sheet steel.

The falsework supporting the lower-deck forms consisted of bents of posts under each cross-girder, braced together transversely and longitudinally. A short longitudinal cap was placed on each post with small diagonal braces. The bents were built complete with



ELEVATION OF ARCH SPAN
SHOWING
FALSEWORK FOR ARCH RIB

Fig. 14.

FIG. 15.—UPPER DECK, TWELFTH STREET TRAFFICWAY VIADUCT, DURING CONSTRUCTION.

FIG. 16.—METHODS OF BUILDING HAND-RAIL, TWELFTH STREET TRAFFICWAY VIADUCT.

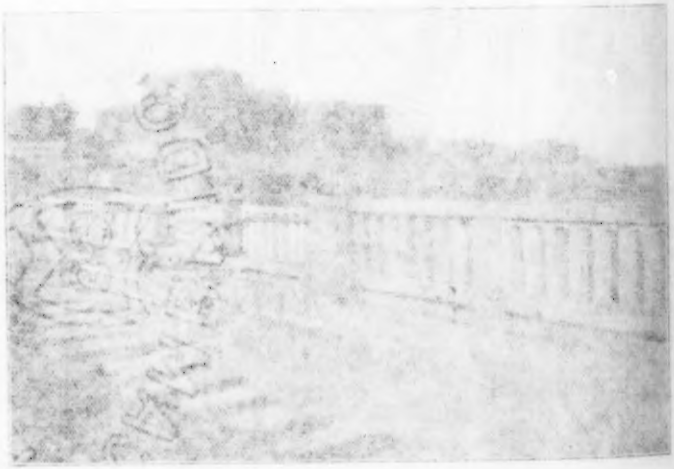


FIG. 17.—SANTA FÉ ARCH SPAN: ROCKER BENTS AND ARCH-RIB GUSSET-PLATE. TWELFTH STREET TRAFFICWAY VIADUCT.

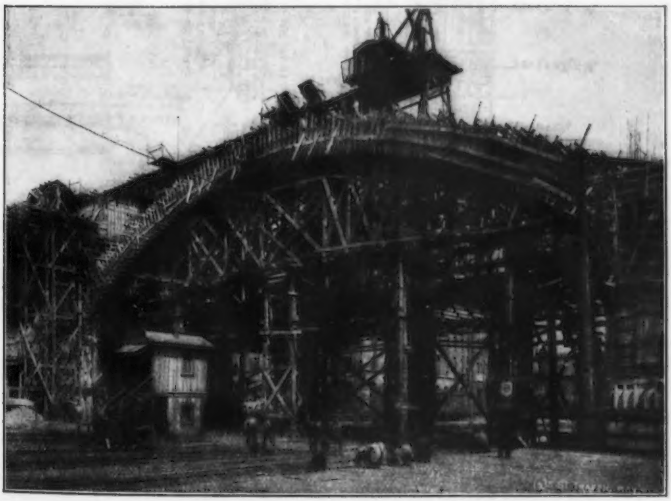
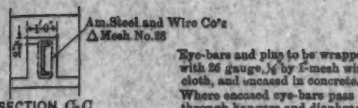
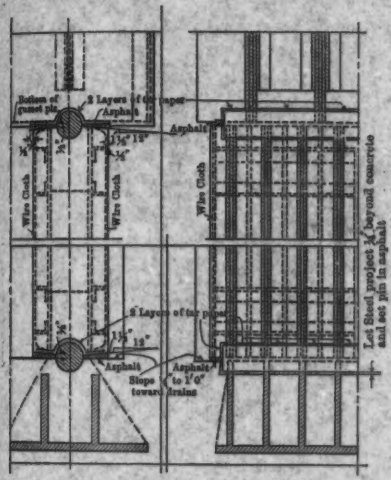


FIG. 18.—FALSEWORK AND FORMS, SANTA FÉ ARCH SPAN, TWELFTH STREET TRAFFICWAY VIADUCT.

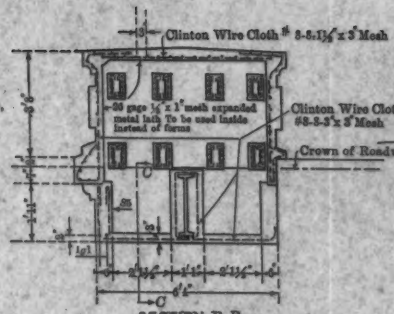


SECTION C-C
Showing encasement of ℄ diaphragm at centre of panels

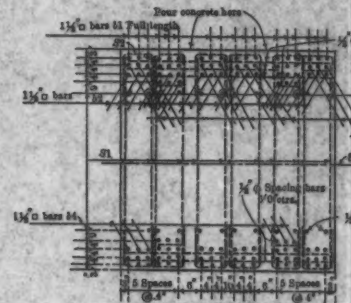
Eye-bars and pins to be wrapped with 26 gauge, ½" by 1"-mesh wire cloth, and encased in concrete. Where encased eye-bars pass through hangers and diaphragms, cover casing with two layers of tar paper



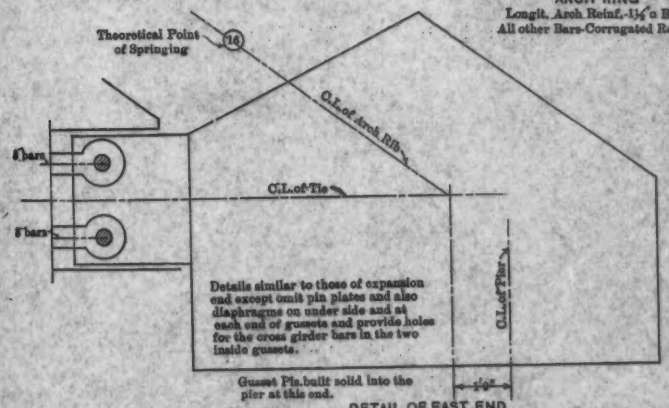
SKETCH SHOWING CONCRETING AROUND PINS



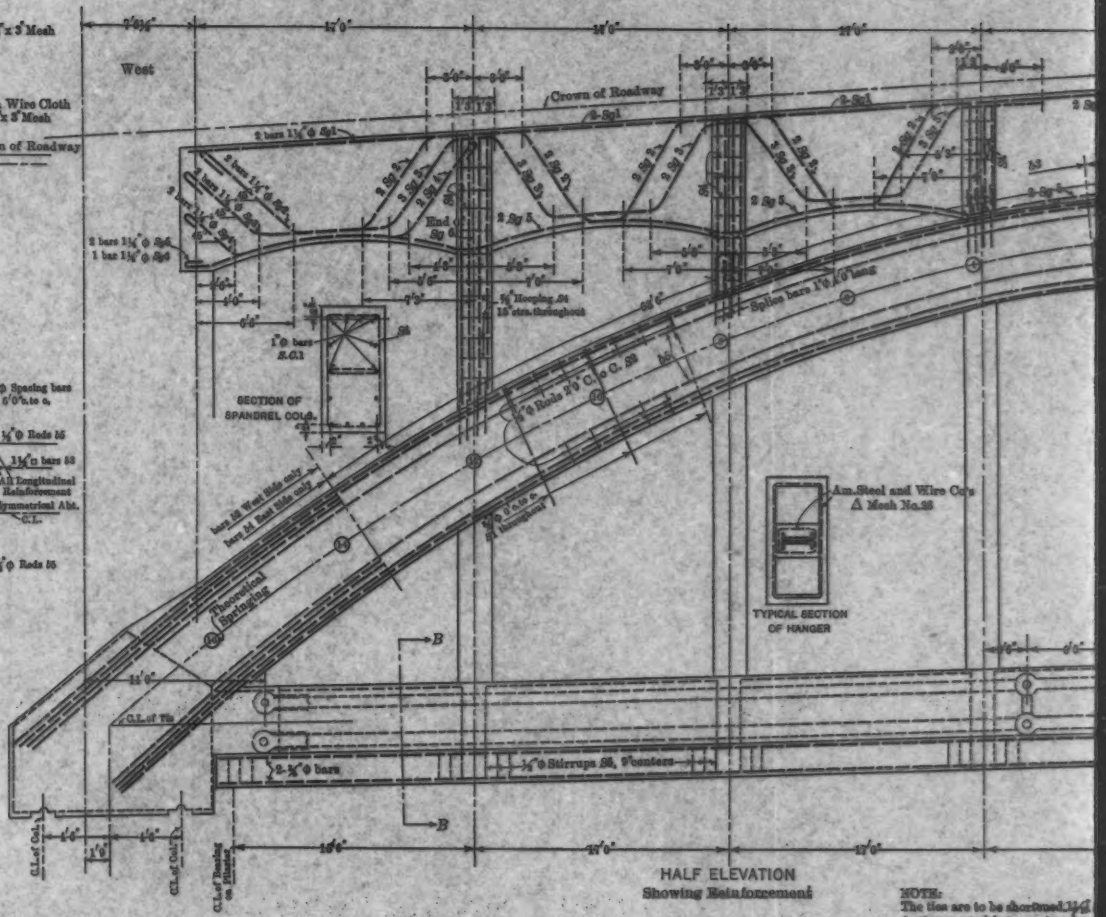
SECTION B-B
Showing tie and Longitudinal Girder



TYPICAL SECTION OF ARCH RING
Longit. Arch Reinf.-1¼"□ Bars
All other Bars-Corrugated Rounds



DETAIL OF EAST END



HALF ELEVATION
Showing Reinforcement

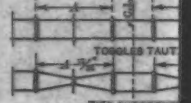
GENERAL NOTES:-

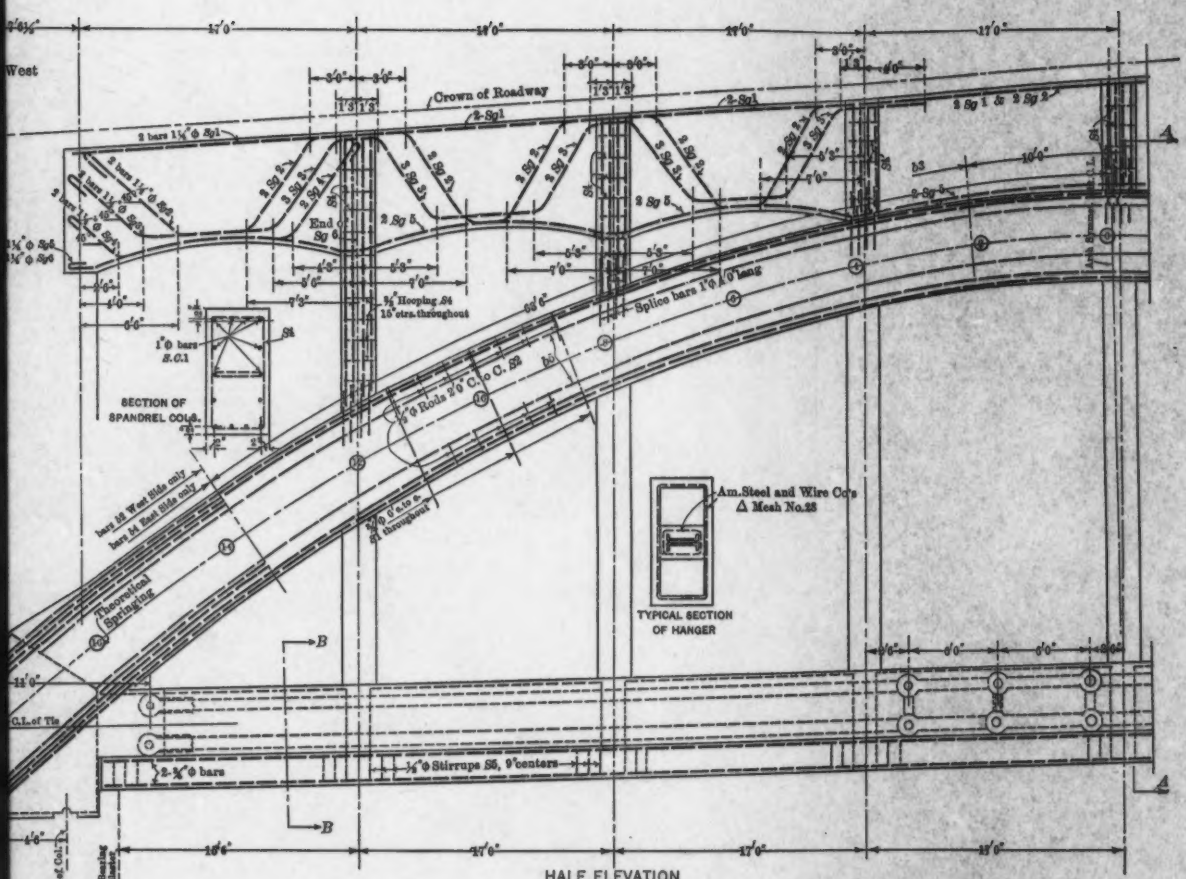
Except where otherwise noted the minimum spacing shall be 3 diameters of the bar
" " " " " = edge distance shall not be less than 2 dia. of the bar. (C.L. of bar to edge of concrete)
" " " " " = bars shall be bent to a radius of not less than 10 dia.
" " " " " = hooks must be of dimensions as shown in this sketch

$R = 2\frac{1}{2}$ diameters of the bar $A = .5$ diameters of the bar

For splicing, the bars must be overlapped 40 diameters, and wired securely together
The splices must be alternated, so that in no case more than half the number of bars are spliced at the same point. Where possible the splices should be located at points of minimum tension

NOTE:
The ties are to be shortened 1¼ [illegible] test, measured between toggle p[illegible] sketches, or such an amount that [illegible] between springing points, after [illegible] the arch, is ¾" shorter than [illegible]
Rivets ⅞" ℗ — All holes sub punc[illegible] to 15/16





epd where otherwise noted the minimum spacing shall be 3 diameters of the bar
" " " " edge distance shall not be less than 2 dia. of the bar. (C.L. of bar to edge of concrete)
" " " " bars shall be bent to a radius of not less than 15 dia.
" " " " hooks must be of dimensions as shown in this sketch
$R = 2\frac{1}{2}$ diameters of the bar $A = 5$ diameters of the bar
splicing, the bars must be overlapped 40 diameters, and wired securely together
splices must be alternated, so that in no case more than half the number of bars
spliced at the same point. Where possible the splices should be located at
ts of minimum tension

NOTE:
The ties are to be shortened $1\frac{1}{16}$" after being made taut, measured between toggle pins as indicated in sketches, or such an amount that the distance between springing points, after dead load is on the arch, is $\frac{3}{16}$" shorter than constructed.
Rivets $\frac{7}{8}$" ϕ — All holes sub punched and reamed to $\frac{15}{16}$"

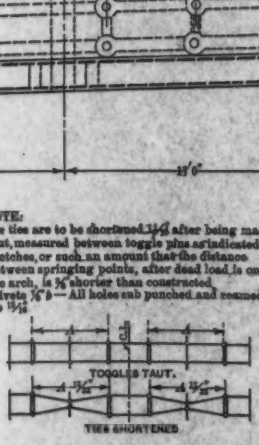
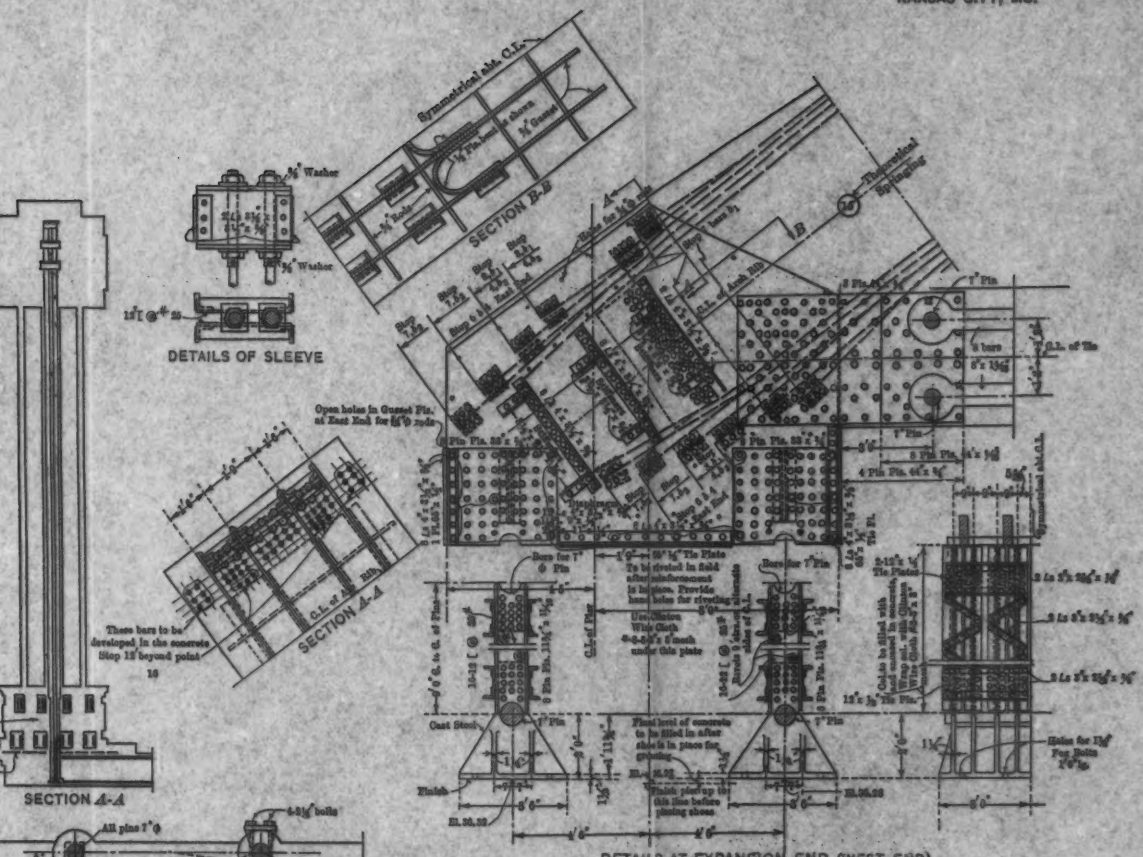
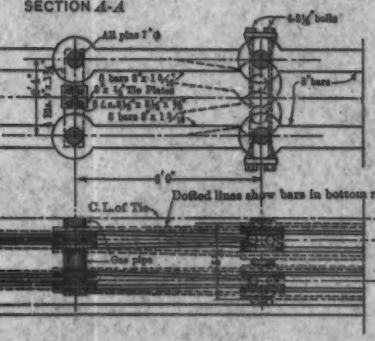
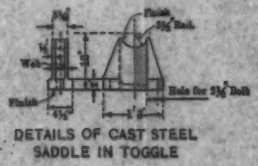


PLATE XXII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXX, No. 1357.
HOWARD ON
TWELFTH ST. TRAFFICWAY VIADUCT,
KANSAS CITY, MO.







DETAILS OF
ARCH SPAN
REINFORCEMENT,
AND
STRUCTURAL PARTS
TWELFTH STREET
TRAFFICWAY VIADUCT
KANSAS CITY, MO.

bracing on the ground and set up with the derrick traveling on the lower deck. The posts were generally 6 by 6-in. timbers, supported on ordinary ground-sills, and the bracing was of 3 by 6-in. pieces bolted in place.

The falsework supporting the upper deck forms was of similar bents resting on the lower-deck floor-slabs, with the posts under the main girder supported on the main girders below. These were made in three parts; a central bent of three posts and bracing supporting the central portion of the cross-beam, and two side bents of three posts each with a cap cantilevered to each side, one end of the cap supporting the cantilever beam and the other extending far enough under the cross-beam to carry a corresponding load, balancing the entire cap load over the posts. These three parts of each bent were prepared on the lower deck near position, or were carted out to position on wagon trucks and raised by the derrick on the upper deck.

The requirements of horizontal clearance for the railroad tracks on Santa Fé Street made necessary the use of three pairs of small wooden truss spans to support the forms for the arch ribs. These trusses rested on bents of posts set in available spaces between the tracks. The arch centering was placed on top of them, and the lower deck forms were suspended from them.

Falsework and forms for about seven girder spans were provided and carried forward as it was removed, the posts of the falsework being spliced as the height increased.

Procedure.—The falsework, forms, and pouring were each, relatively, on the lower deck about two span lengths ahead of the upper deck, as the falsework for the upper deck could not be placed on the lower-deck concrete until it had hardened somewhat. The whole construction thus moved forward in sequence, maintaining about the same relation of lower deck advance. This sequence of operations is shown by Fig. 20.

The reinforcing bars were delivered, as far as possible, in lengths correct for use. Diagrams were prepared showing the shape and dimensions for bending each bar and assigning a mark to it. The bars were bent in the yard, and each special bar was tagged with its erection mark on a linen tag. Groups of plain bars, etc., were not tagged individually.

As the forms neared completion, the prepared bars were carried out on the tram cars and set in place. Metal clips, separators, and wire ties at the crossing of every two bars, supported them in place. Use was made of small cement briquettes or blocks in some cases to hold the bars up from the forms.

The placing of bars and depositing of concrete in the columns was carried forward in customary routine. For the upper deck, all the bars were placed for main girders, cross-girders, cantilevers, and slabs complete from the last division to a bulkhead placed across the structure at a convenient point. Concrete was poured continuously across the whole width of the structure, filling the girders, beams, slabs, etc., without a stop, for the longitudinal distance selected.

The concrete was transported in cars, as described, hauled by cable, discharged into a movable distribution hopper, and from this into Insley chutes, which conveyed it to final position in the forms. A trussed steel chute about 80 ft. long carried the concrete from the receiving hopper on the upper deck forward to position in the lower portions of the structure. The concrete was mixed to a soft consistency so that it flowed readily, but it was not so wet as to have water standing on its surface. It was spaded and sliced at the forms and around the reinforcing bars to insure complete contact, and to work forward the mortar.

The maximum capacity of the concrete plant delivering 1 300 ft.—or, say, to the mid-point of the structure—was about 30 cu. yd. per hour. The largest quantity placed in continuous run was about 450 cu. yd. in 15 hours. A trip one way, from the plant to the east abutment, required 3½ min.

The hand-rail units were cast on the deck in metal-lined wooden forms, near the place where they were to be used. The lower rail was cast in place, the separately cast balusters were set in it in rows, and the pre-cast top rail was set on them, extending into the posts. The forms for the posts, except for the capital, were clamped on, and the posts were cast in place. The stairs were cast in units of one riser and one tread. These were supported on falsework and projected into the forms for the supporting girders, which were built in place.

Materials and Wages, General Observations.—The total quantity of timber used on the job was about 1 500 000 ft. b.m. This may be

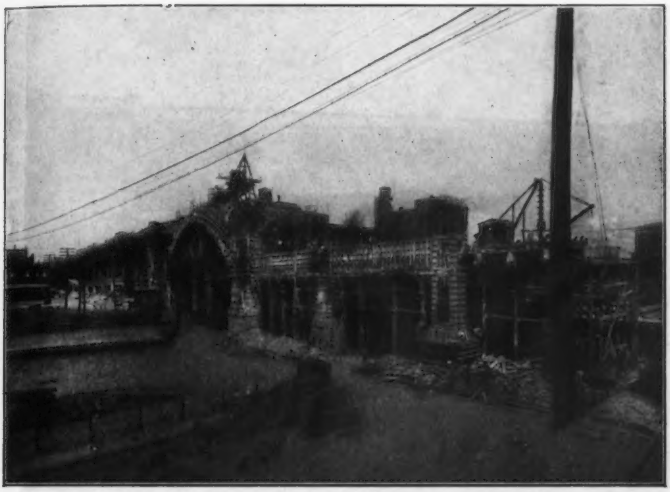


FIG. 19.—LOWER DECK ADJACENT TO ARCH SPAN, TWELFTH STREET TRAFFICWAY VIADUCT, DURING CONSTRUCTION.

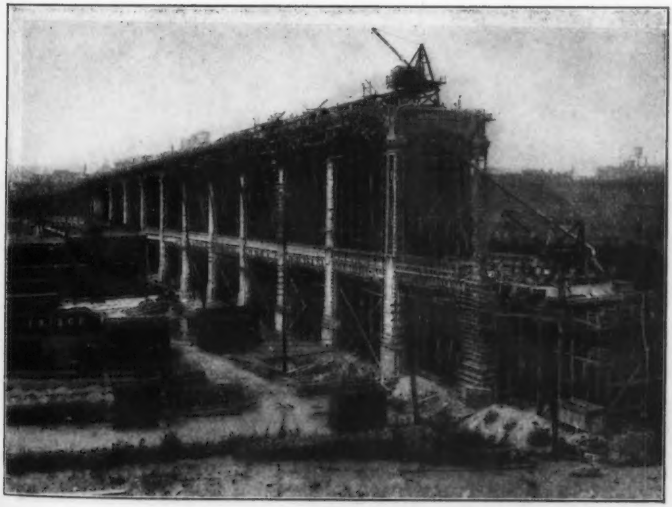


FIG. 20.—TWELFTH STREET TRAFFICWAY VIADUCT: SEQUENCE OF OPERATIONS DURING CONSTRUCTION.

FIG. 21.—SANTA FÉ ARCH SPAN AFTER COMPLETION, TWELFTH STREET TRAFFICWAY VIADUCT.

FIG. 22.—TWELFTH STREET TRAFFICWAY VIADUCT, LOOKING EASTWARD ON THE LOWER DECK.

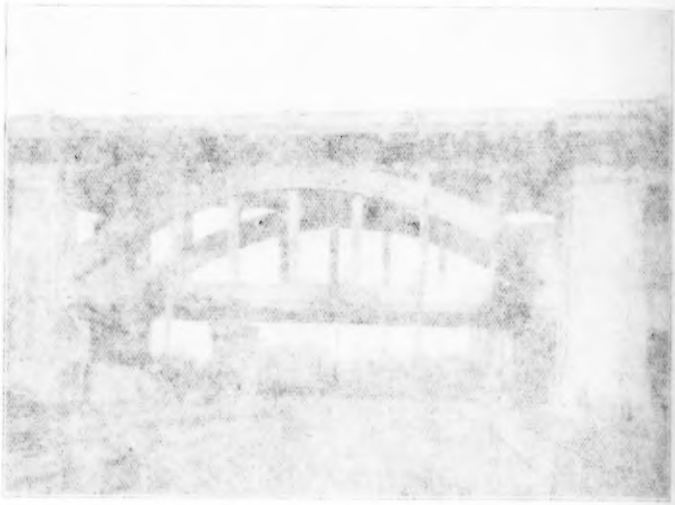
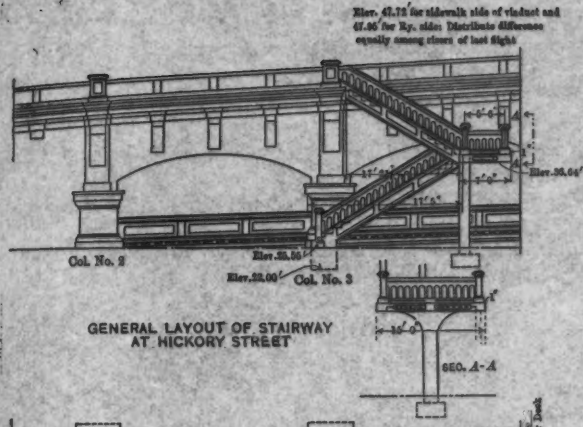
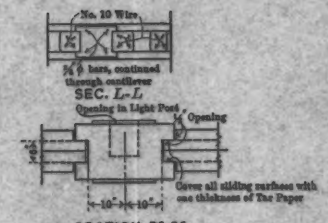


PLATE XXIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXX, No. 1357.
HOWARD ON
TWELFTH ST. TRAFFICWAY VIADUCT,
KANSAS CITY, MO.

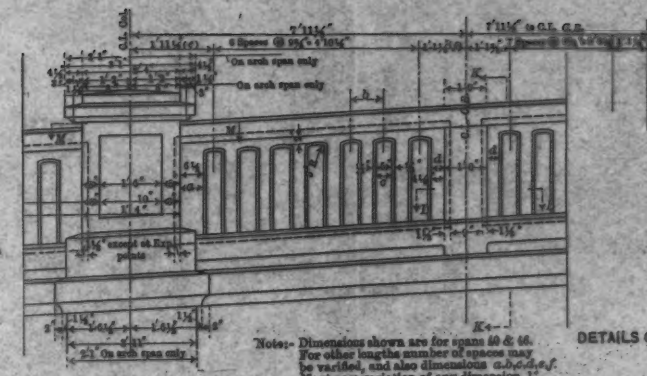


GENERAL LAYOUT OF STAIRWAY AT HICKORY STREET

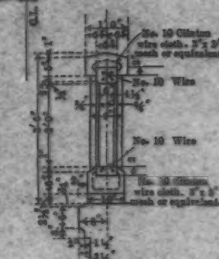
SEC. A-A



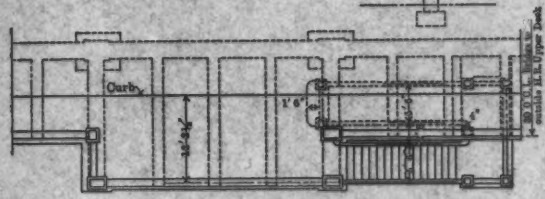
SECTION M-M
SHOWING EXPANSION JOINT FOR HAND-RAIL

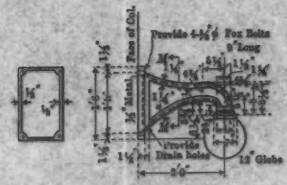


Notes:- Dimensions shown are for spans 40 & 66. For other lengths number of spaces may be varied, and also dimensions a, b, c, d, e, f. Maximum variation of any dimension, 1'

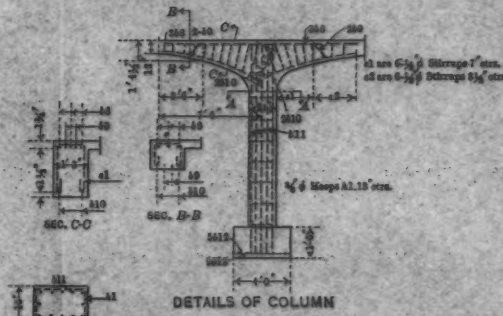


SECTION K-K
DETAILS OF UPPER DECK HAND-RAIL AND FASCIA GIRDER

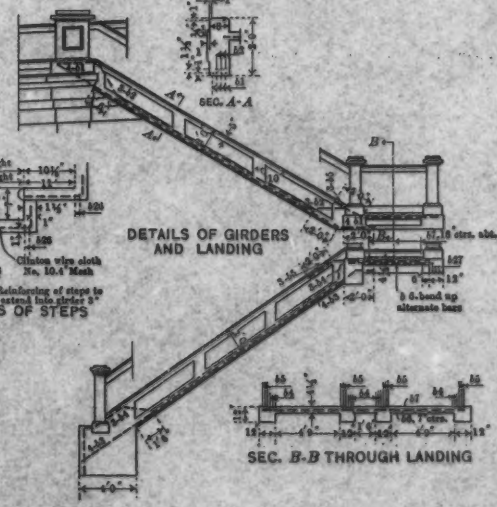




DETAIL OF CAST-IRON LAMP BRACKET FOR LOWER DECK



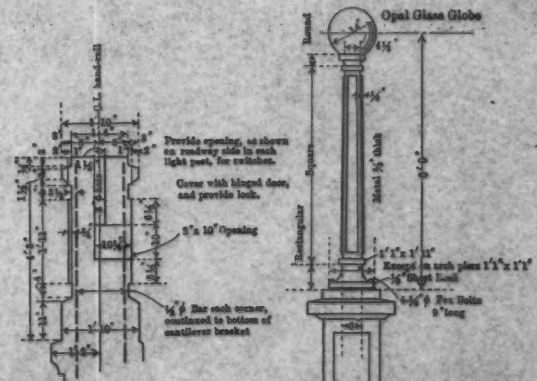
DETAILS OF COLUMN



DETAILS OF GIRDERS AND LANDING

DETAILS OF STEPS

SEC. B-B THROUGH LANDING



CROSS-SECTION OF LIGHT POST (AT C.L.)
OTHER MAIN POSTS SIMILAR, BUT WITHOUT OPENINGS.

DETAIL OF CAST-IRON LAMP STANDARD UPPER DECK

DETAILS OF
HAND-RAILS, STAIRWAYS,
LIGHT BRACKETS, ETC.
TWELFTH STREET TRAFFICWAY VIADUCT
KANSAS CITY, MO.

roughly divided into 600 000 ft. of form lumber, 850 000 ft. of falsework, and 50 000 ft. in general use, including bins and mixing plant. The average cost of this material was about $20 per thousand, or about $1 per yd. applied to the whole concrete, or $1.25 per yd. applied to concrete above the bases.

The number of men employed varied from 150 to about 325, an average of about 200 men, of whom, perhaps, 150 were carpenters and helpers. The wages prevailing were, for common labor, $2.00, for carpenters' helpers, $2.50, for carpenters, $3.50, for finishers, $3.50, and for derrick and hoist operators, $4.00; all for 9 hours per day.

The conduct and execution of the work merit special comment. The viaduct was built substantially in the allowed time of 15 months, moving forward in logical and orderly manner, largely due, of course, to systematic handling of labor and materials. On the first day of every week, a schedule of work to be accomplished each day of that week was prepared in writing, and the foremen were instructed, not only what to do each day, but also how much had to be done. If the amount assigned was not completed at the usual stopping time, the men kept at work until it was done. This applied particularly to placing concrete, the day's run being completed more than once after midnight. The conditions surrounding the work permitted it to be handled as a manufacturing proposition, and with factory-like precision and system.

There was some difficulty in securing stone as clean as desired, for the limestone strata in this region are overlaid with clay and other objectionable materials; and, especially during a rainy season, it is impossible to prevent some clay and soft shale from mingling with the stone in the crusher bins. In fact, the more careful crushers shut down during rainy weather to avoid this difficulty. The stone cost $1.15 per cu. yd., delivered in cars at the site. The sand supply from the Kaw River is clean, sharp, and well graded, testing about 52% retained on a No. 30 sieve. Its cost delivered in cars at the site was about 65 cents per cu. yd. The cement was Kansas City-Portland, manufactured in a mill about 15 miles from the site, and costing, delivered in cars at the site, $1.20 per bbl. (plus sack loss). These delivered prices include somewhat complex switching charges.

Reinforcing steel delivered cost about 1.75 cents per lb.

6.—Quantities and Costs.

The contract assigns unit prices for forty classified items, including many classifications of concrete with regard to location in the structure. The following summarized figures were prepared from such classifications, and an attempt has been made to give the salient features of the final quantities of the structure, as built. These costs are the amounts paid by the City, and include the contractor's profit.

There are about 32 500 cu. yd. of concrete, distributed about as follows:

	Cubic yards.
Concrete piles	1 360
Columns, piers at Santa Fé Street, and footings	11 300
Upper deck, including girders, cross-girders, cantilevers, floor-slabs, etc.	11 300
Lower deck, including girders, cross-girders, floor-slabs, etc.	3 600
Retaining walls and abutments	3 000
Ribs of arch at Santa Fé Street	600
Hand-rails	500
Stairways, sidewalks, curbs, paving base on fills, and miscellaneous	800

No special payment was made for excavation, but the amount paid for concrete included its cost. The total excavation was about 10 000 cu. yd. The amounts paid for concrete in place varied from about $8.00 to $13.50 per cu. yd. for different classifications, averaging $9.45 for the whole structure, omitting only the piles and the hand-rails, as both of these were paid for at linear foot prices.

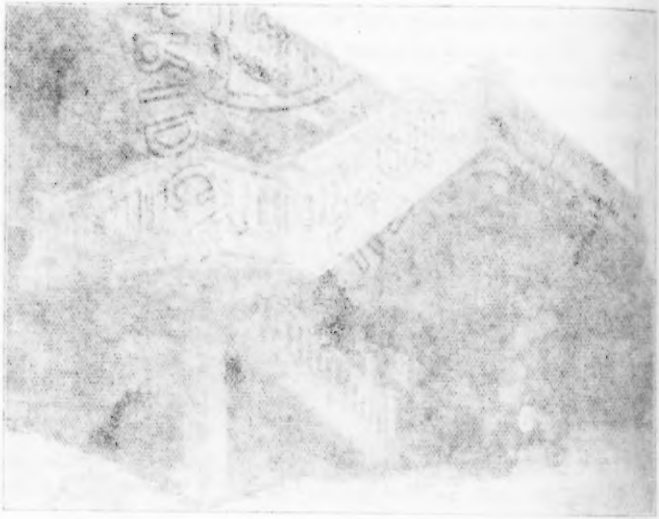
There were driven 2 610 piles, having a total length below bottoms of footings of about 47 000 lin. ft. and an average length each of 18 ft., costing in place $1.20 per lin. ft.

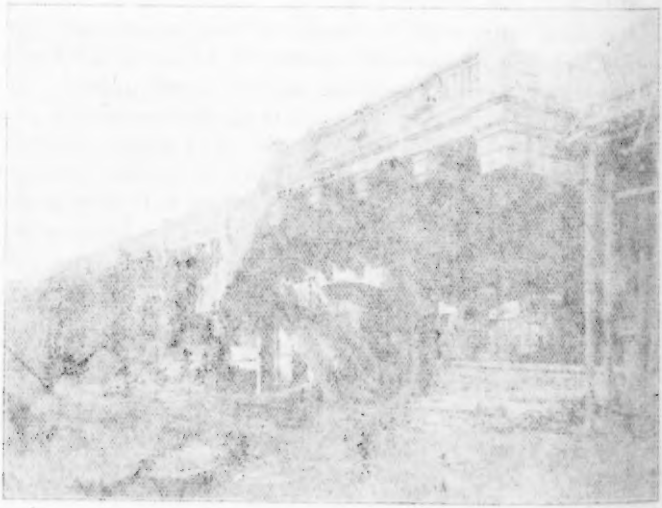
The approximate quantities of metal, and the prices paid for it in place are as follows:

Reinforcing steel, ¾ in. in diameter and greater	2 900 000 lb. at 2.95 cents per lb.
Reinforcing steel, less than ¾ in. in diameter	1 000 000 lb. at 3.25 cents per lb.
Unfabricated metal, such as bed and bearing plates, guard-angles, etc.	100 000 lb. at 4.40 cents per lb.
Fabricated structural metal	620 000 lb. at 4.60 cents per lb.
Cast-iron lamp-posts and brackets	25 000 lb. at 5 cents per lb.

FIG. 23.—STAIRWAY AT HICKORY STREET, TWELFTH STREET TRAFFICWAY VIADUCT.

FIG. 24.—STAIRWAY AT MULBERRY STREET, TWELFTH STREET TRAFFICWAY VIADUCT.





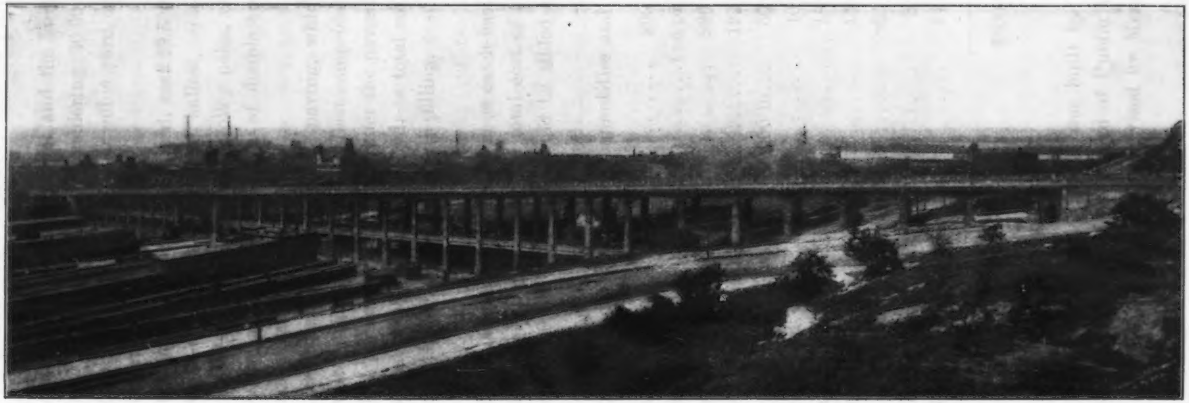


FIG. 25.—TWELFTH STREET TRAFFICWAY VIADUCT, KANSAS CITY, MO.

Omitting the fabricated structural metal and the lamp-posts gives a total of 4 000 000 lb. of metal, or, considering 30 600 cu. yd. of concrete, an average of 130 lb. of metal per cubic yard, at an average price of 3.06 cents per lb.

The lighting system, complete as described, cost $9 500.

The double street-car tracks, including ballast, cross-ties, track rails, inner steel guard-rails, bonding, and trolley poles, cost, in place, a total of $18 000 for a length of 2 600 ft. of double track, or, say, $3.50 per lin. ft. for single track.

There are 14 500 sq. yd. of wood-block paving, which cost $2.25 per sq. yd. The water-proofing under the ballast comprised about 5 000 sq. yd., costing $1.71 per sq. yd., and that under the pavement included 11 500 sq. yd. at a cost of 58.5 cents per sq. yd.—a total cost of $15 000.

There were some 40 000 cu. yd. of earth filling, costing 28 cents per cu. yd.

There were other miscellaneous items such as catch-basins, drainage pipes, sewer connections, etc., bringing the total cost of the structure to the city to about $600 000, to which should be added some $40 000 for engineering.

The following is a summarized table of quantities and costs:

Concrete piles (2 160)	$60 000
Excavation, about 10 000 cu. yd.	(no price)
Concrete, 30 600 cu. yd.	290 000
Reinforcing metal, 4 000 000 lb.	122 000
Structural metal and castings, 645 000 lb.	30 000
Lighting system	10 000
Street-car tracks, etc.	18 000
Water-proofing	15 000
Block pavement	33 000
Hand-rails	8 000
Earth filling and miscellaneous	14 000
	$600 000

7.—Personnel.

The Twelfth Street Trafficway Viaduct was built by the City of Kansas City, Mo., under control of the Board of Public Works. The plans were made and the construction supervised by Messrs. Waddell

and Harrington, Consulting Engineers, under the personal supervision of Mr. Harrington. The writer prepared the general preliminary studies and estimates, directed the design, and wrote the specifications. L. R. Ash, M. Am. Soc. C. E., was City Engineer at the time the plans were adopted, and after joining the firm, directed the construction work. The details of design were largely the work of Messrs. A. E. Slettum and C. W. Yelm. Mr. M. J. Maher was Resident Engineer, in charge of the entire construction work.

The structure was built under one general contract by The Graff Construction Company, of Seattle and Kansas City. Mr. Graff gave the work his constant personal attention, and was ably assisted by his superintendent, Mr. Harold E. Ketchum, in handling the building operations and in the design of the contractor's plant, falsework, forms, and other temporary structures; Mr. George Hockensmith was General Foreman.

For their courteous assistance in the preparation of these data, the writer desires to express his cordial appreciation to the above gentlemen and to H. C. Tammen, Assoc. M. Am. Soc. C. E., at present Designing Engineer of Harrington, Howard, and Ash.

DISCUSSION

F. W. GREEN,* ASSOC. M. AM. SOC. C. E. (by letter).—The author has made a noteworthy contribution to engineering literature. Unique seems to be the word best adapted, adjectively, to apply to a structure which is neither straight as to alignment, level as to gradient, symmetrical as to transverse loading, nor orthodox as to the tenet of design which requires that indeterminate stresses must be solved by being avoided. Mr. Green.

Moreover, it is unique in that the embellishments of art have been invoked, with very gratifying success, to commend to the esthetic emotions a structure primarily intended to be useful. This, too, under circumstances most unpropitious—an ugly *situs*, an unbalanced subject, and an inauspicious environment; and, as the author states, "without any extra funds for adornment".

A consideration of the problem creates a feeling of dismay, which is superseded later by one of admiration for the ingenuity shown and the resource developed in mastering the many baffling features. To the mind mathematically inclined, the graphical method for solving the integrals shown on pages 494 and 495 will commend itself both for elegance and simplicity. The method for obtaining moments and deflections for the rectangular frames (given on page 496) by the solution of ten simultaneous equations, though no laughing matter, offers opportunity for mirth: "When a structure is so formed that the stresses in it cannot be determined by statics, it is said to be a redundant system; and the principle of least work applicable to it is as follows, etc."† Do we not have here a paradox? Is it strictly proper to say that the solution of these ten simultaneous equations involves the principle of least work? The author says: "By arranging these * * * in tabular form, the solution is found with comparatively little labor." Did he intend "little" in the sense of *"multum in parvo"*?

Seriously speaking, however, the impression uppermost in the writer's mind, after reading this paper, is that this work marks another important instance of the present tendency in American development to pass from the stage of pioneer civilization, with its temporary, but economical and expedient, types of buildings, works, ways, and systems, to an era of permanence, beauty of line and mass, sufficiency, and efficiency.

The design of this structure is a bold departure in many respects from the established canons of design, and the novice should be admonished not to undertake liberties of this kind unless and until he is absolutely sure of his ground. For example, a comparatively slight settlement of the foundation under one of the 120-ft. columns conceivably might have disastrous consequences.

* Stamps, Ark.
† Merriman's "Mechanics of Materials", 10th Ed., p. 321.

Mr. Mensch.

L. J. MENSCH,* M. AM. SOC. C. E. (by letter).—Reinforced concrete seems to enlist the brightest engineers under its colors. Though viaducts like that described in this paper have often been built of steel, one would search American literature in vain for a description of such viaducts in which the attempt was made to design the structure for temperature and other so-called secondary stresses, as was done for the Twelfth Street Trafficway Viaduct of Kansas City. Because such refined—but often very necessary—computations are rare in American practice, we need not be surprised that, in one of the first attempts, such computations lack the simplicity, elegance, and surety of the

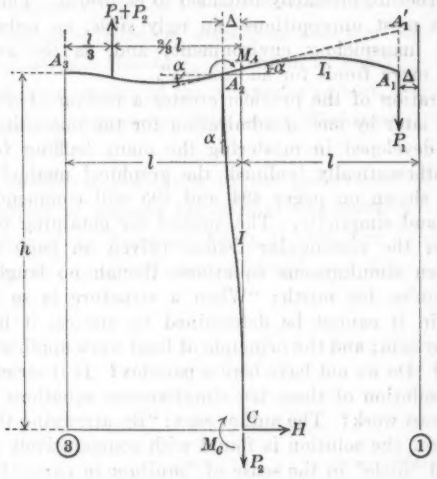


FIG. 26.

schooled designer. Not until a great many of our students take post-graduate courses in the design of so-called "higher structures" can a material change be expected in this matter. Take, for example, Fig. 2 and its explanation. The writer has designed indeterminate structures for more than twenty years, yet he can follow the reasoning only with great difficulty. There is no need for such lack of clearness.

The problem can be solved entirely with much less effort, by finding the unknown conditions of the supports at A_1 and C (Fig. 26). A sliding joint was assumed at A_1, hence the only unknown that can act there is the vertical force, P_1. The point, C, is fixed, hence the support will exert on the column, A_2 C_1, a thrust, H, a pull, P_2, and a moment, M_C.

* Chicago, Ill.

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These unknown reactions can easily be found by the rules for the change of angles and for the deflection of a beam:

$$\Delta\alpha = \int \frac{M ds}{E I} = \frac{1}{E I} \text{ (area of diagram of moment)} \ldots\ldots\ldots\ldots (1)$$

$$\Delta y = \int \frac{M x ds}{E I} = \frac{1}{E I} \text{ (static moment of diagram of moment about the point, the deflection of which is sought)} \ldots (2)$$

For the force, P, acting at the end of a cantilever,

$$\Delta\alpha = \int \frac{P x dx}{E I} = \frac{P l^2}{2 E I}$$

$$\Delta y = \int \frac{P x \times x dx}{E I} = \frac{P l^3}{3 E I}, \text{ provided } I \text{ is constant.}$$

$A'_1 A_1$ can be considered the deflection of the point, A_1, due to P_1, or

$$l\alpha = \frac{P_1 l^3}{3 E I_1},$$

or

$$\alpha = \frac{P_1 l^2}{3 E I_1} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

It is true, as the author has stated, that the point of inflection of $D B$ is $\frac{1}{3}\ l$ from D, and, in order to preserve the equilibrium, we can imagine that P_1 and P_2 are acting at this point, producing a moment, $(P_1 + P_2)\ \frac{l}{3}$, at A_3, and $(P_1 + P_2)\frac{2}{3}\ l$, at A_2.

Then

$$\alpha = \frac{(P_1 + P_2)\frac{2 l}{3} \times \frac{2 l}{3} \times \frac{1}{2} - (P_1 + P_2)\ \frac{l}{3} \times \frac{l}{3} \times \frac{1}{2}}{E I_1}$$

$$= (P_1 + P_2)\frac{l^2}{6 E I_1} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

From Equations 3 and 4, $\frac{P_1 + P_2}{6} = \frac{P_1}{3}$,

or

$$P_1 = P_2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

We can determine, also, from the column, $A_2\ C$,

$$\alpha = \frac{M_c h - \frac{H h^2}{2}}{E I} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

and from Equations 3 and 6, $\frac{P_1 l^2}{3 E I_1} = \frac{M_c h - \frac{H n^2}{2}}{E I}$

Mr. Mensch. let $n = \frac{h}{l}\frac{I_1}{I}$, $\qquad M_c\, n - \frac{H h\, n}{2} = \frac{P_1 l}{3}$ .....(7)

From $A_2 C$,

$$\Delta = \frac{\frac{M_c h^2}{2} - \frac{H h^3}{6}}{E I}, \text{ or } M_c - \frac{H h}{3} = \frac{2 E I}{h^2}\Delta \text{ .....(8)}$$

By taking moments about A_2,

$$(P_1 + P_2)\frac{2}{3} l + P_1 l + M_c = H h \text{ .....(9)}$$

Equations 7, 8, and 9 have three unknowns, and we easily obtain

$$\left.\begin{aligned} H h &= \frac{2.4\, E I}{h^2}\Delta \frac{1 + 7 n}{1 + 2.8 n} \\ M_c &= \frac{2.4\, E I}{h^2}\Delta \frac{1 + \frac{7}{2} n}{1 + 2.8 n} \\ P_1 l &= 3.6\, E \frac{I}{h^2}\Delta \frac{n}{1 + 2.8 n} \\ M_A &= 8.4\, E I \times \frac{\Delta}{h^2} \times \frac{n}{1 + 2.8 n} \end{aligned}\right\} \text{ .....(10)}$$

If the structure is cast monolithically, without any joints, we can place $I = \frac{b \times d^3}{12}$.

The moment of resistance $= \frac{b \times d^2}{6}$, and

the stresses at $C = \frac{M_c}{\frac{b \times d^2}{6}} = \frac{2.4\, E \frac{b \times d^3}{12}}{\frac{b \times d^2}{6}} \frac{\Delta}{h^2} \frac{1 + \frac{7}{2} n}{1 + 2.8 n}$

$$= 1.2\, E\, d \frac{\Delta}{h^2} \frac{1 + \frac{7}{2} n}{1 + 2.8 n} \text{ .....(11)}$$

The moment of inertia of the columns varies, and, assuming the section of the base only to govern, $n = 4.46$; of the top only to govern, $n = 2.76$, which produces hardly any difference in the value of the fraction, $\frac{1 + \frac{7}{2} n}{1 + 2.8 n}$, as the respective quotients are 1.23 and 1.233.

Assuming $d = 7$ ft., $l = 48$ ft., $h = 89$ ft., $E = 3 \times 10^6$, $\Delta = \frac{l}{2\,000}$, then $S_c = 94$ lb. per sq. in.

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Where the bents are $h = \frac{89}{4} = 22$ ft. 6 in., S_c is nearly sixteen times as great, or 1 504 lb. (the value of $\frac{1 + \frac{7}{2} n}{1 + 2.8 n}$ changing only to 1.2), which is not counterbalanced by the dead load of the structure, and large cracks will be prevented only by steel reinforcement.

In this case, $E I$ must be replaced in Equation 10 by $E_s A_s d'^2$, where E_s is the modulus of elasticity of the steel, A_s is the steel reinforcement, in square inches, on the tension side, and $d' = jd$, the moment of resistance being represented by $A_s jd$.

The stress in the reinforcement at C

$$= \frac{2.4 E_s A_s d'^2 \frac{\Delta}{h^2} \frac{1 + \frac{7}{2} n}{1 + 2.8 n}}{A_s d'} = 2.4 E_s \frac{d'}{h^2} \Delta \frac{1 + \frac{7}{2} n}{1 + 2.8 n} \quad \ldots\ldots(12)$$

In this latter case, M_c is of considerable magnitude, and will cause an unequal loading of the piles, the latter being symmetrically arranged around the columns.

Evidently the author forgot to assume a thrust at C, or he would have found that M_2, in Fig. 3 must be negative, if he considers M_1 positive.

It is absolutely unnecessary to resort to the summation methods, where the expression, $\frac{d s}{I}$, varies.

In most cases we can put $\frac{d s}{I} = \frac{d x}{I}\left(1 + C\frac{x}{l}\right)$, which will allow of easy integration of $\frac{M d s}{I}$ and of $\frac{M x d s}{I}$.

It is extremely rare, where, instead of a straight-line variation, a parabolic, or hyperbolic variation of $\frac{d s}{I}$ must be adopted.

The advantage of integration over summation is evident by the fact that in the first case we can obtain a formula of general application, but in the latter we obtain the solution of one particular problem only.

The use of the formulas for continuous beams for solving stresses in framed structures is accompanied with great difficulties, and is not the shortest or most lucid solution of the problem.

Any one reading the author's method of finding the wind stresses in a two-story frame, would think it requires the enormous labor of solving a six times indeterminate structure, even if both columns have the same moment of inertia. Such a structure is only twice indeterminate. On account of the symmetry of construction, the point of inflection of the girders must be the center of the span. By the same

Mr. Mensch.

method as the writer used for the temperature stresses, the following equations may be found to solve for P_1 and P_2:

$$\frac{I_1}{I_2} P_2 = P_1 + 6\, n_1 \times P_1 - 6\, \frac{M'\, n_1}{l} \quad \ldots\ldots\ldots\ldots (13)$$

$$0 = P_2 + 6\,(P_1 + P_2)\, n_2 - 6\, \frac{M''\, n_2}{l} \quad \ldots\ldots\ldots\ldots (14)$$

When $n_1 = \frac{h_1}{l}\,\frac{I_1}{I}$, $n_2 = \frac{h_2}{l}\,\frac{I_2}{I'}$, and $M' = \frac{H_1 h_1}{2}$, and $M'' = H_1\left(h_1 + \frac{h_2}{2}\right) + H_2\,\frac{h_2}{2}$.

Where the moments of inertia of the columns are not alike, the exact solution, as above stated, is very laborious, and mostly time and men are lacking to do such work; but, fortunately, we can find an approximate solution, by considering that P_1 and P_2 vary only slightly from the values obtained from Equations 13 and 14, and that the point of inflection of the girders moves only very slowly from the center toward the weaker column. Where the ratios of the moments of inertia of the columns is infinite, the point of inflection would be at the connection of the girder and the weaker column; in the Twelfth Street Trafficway Viaduct it is about $\frac{3}{10}\, l$ from the weaker column. The assumption of the point of inflection is easily checked by calculating the deflection at the top of the frame. The deflection of the left column must be equal to that of the right, and if at the first trial they are not alike, a good guess will serve to make them so.

By comparing Fig. 27 with Fig. 7 it should be noted that the columns show one point of inflection at B and one midway between B and C, which were omitted by the author.

The equation for the funicular line of an arch, as shown in Fig. 8, saves considerable time over the graphical method generally used in arches with earth fill, but it is not a very fair approximation in an arch of open spandrel construction. In the latter case it is more correct to assume that $P_s - P_c$, the difference of weight per linear foot at abutment and crown, respectively, decreases to zero according to a straight line from the abutment to the crown, in which case the equation of the funicular polygon is

$$y = \frac{4\,h}{l^2}\,\frac{3\,x^2 + 2\,(u-1)\,\frac{x^3}{l}}{2+u},$$

and the corresponding thrust $= \frac{P_c\, l^2}{8\,h}\left(1 + \frac{(u-1)}{3}\right)$.*

* The equations for both cases might be found in the writer's "Reinforced Concrete Pocketbook", where also the ordinates of the curves are calculated in fractions of h for various values of u.

Mr. Mensch.

The summation method of finding the statically unknown value of the thrust, reaction, and moment of the abutment, requires very lengthy computations, with great likelihood of numerical and other errors. The writer can state positively that such errors must have been made when finding the influence lines, as the apex of the influence line for moments due to vertical loads at Point 6 should have been higher than at Point 8, on Plate XVII; and the apex at Point 0 should have been at least 8% lower than at Point 2.

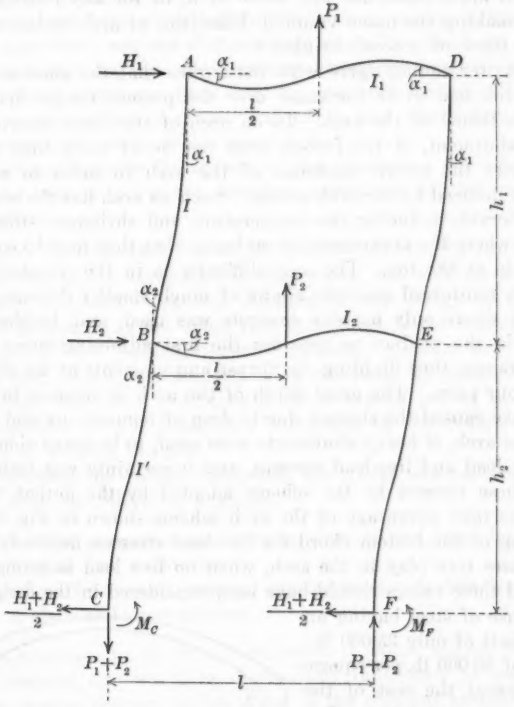

FIG. 27.

It is well known that Howe and many other writers have given very simple formulas for the statically indeterminate values of a parabolic arch with constant $\frac{ds}{I}$; but there nearly all writers have stopped. The question is often asked: can these formulas be applied to arches with varying $\frac{ds}{I}$, and of correct shape to suit the earth fill?

Mr. Mensch.

In most cases they can be applied directly for live loads, but they fail widely for temperature and shrinkage stresses. As indicated before, $\frac{ds}{I}$ may be put $= \frac{dx}{I}\left(1 + C\frac{x}{l}\right)$ and, instead of a parabolic curve, the one given by the author accompanying Fig. 8 can be used. The work necessary to integrate the elastic equations is not greater than the numerical work of the summation method, and the results are formulas of general application, for any value of u, or for any relation of rise to span, making the much vaunted difficulties of arch design nearly as small as those of a common girder.

The writer cannot agree with the author that the abutment of the 130-ft. arch had to be increased over the present design in order to resist the thrust of the arch. In an arch of the shape shown in Fig. 28, the abutment, if reinforced, need not be of more than twice or three times the crown thickness of the arch in order to resist the moments induced by the arch action. Such an arch has the advantages of considerably reducing the temperature and shrinkage stresses over a design where the abutments are so heavy that they may be considered immovable at the top. The only difficulty is in the foundations, but also here, reinforced concrete allows of much smaller dimensions than formerly, where only massive concrete was used, and, besides, it was possible in the viaduct to consider the first adjoining spans at each side as frames, thus dividing the thrust and moments at the abutments among four piers. The great depth of the arch in relation to the rise would have caused the stresses due to drop of temperature and shortening of the arch, if heavy abutments were used, to be many times larger than the dead and live-load stresses, and it certainly was fortunate to reduce these stresses by the scheme adopted by the author, after he omitted to take advantage of the arch scheme shown in Fig. 28. The shortening of the bottom chord for live-load stresses, however, brought new stresses into play in the arch, when no live load is acting on the arch; and their values should have been considered in the design.

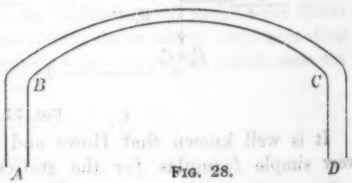

FIG. 28.

The use of steel having an elastic limit of only 35 000 lb., instead of 50 000 lb. per square in., increased the cost of the structure about $40 000. The writer cannot understand the prejudice of engineers against the use of a higher grade of steel, made of billet stock, in reinforced concrete construction. All tests made by the best-known authorities show that the ultimate carrying capacity is increased by its use by more than 50% over low-carbon steel. There certainly is a very small chance that shock

will affect the steel in columns or footings, or that the small steel bars in the slabs will be affected by shocks. The only place where an engineer may properly doubt the advisability of using high-carbon steel reinforcement is in the bent bars of girders, because the bars are often injured by careless bending. Where careful bending is done, with large radii, say about six to ten times the diameter, no engineer need be afraid of any unfavorable results. The writer used high-carbon bars for the reinforced floors of the Utah copper concentrator, in Garfield, Utah, and although about 8 000 tons of material have been vibrating 1 000 000 times a day, without interruption since 1907, not the slightest sign of failure has been detected. Mr. Mensch.

The provision in the specifications requiring bars to be firmly bound and tied together where they lap was unfortunate. The stress in bars is transmitted by shear and bond, and where bars are tied together the bond surfaces are decreased.

The author deserves the thanks of the Profession for offering this paper, and especially for the detailed description of the methods of construction.

HOWARD W. HOLMES,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer is very much interested in this paper, not only on account of the several uncommon features involved in the design of the structure, but because it is not often that structures of the magnitude and importance of the Twelfth Street Trafficway Viaduct are described as fully and clearly, particularly with reference to arch design and construction features. Publications of this kind are of great value to all members of the Profession interested in the design and construction of modern bridges. Mr. Holmes.

In reviewing the details submitted, the writer was particularly impressed with the following features:

First.—The unequal cross-sectional area of the main longitudinal girders and arch ribs;

Second.—The curved tension reinforcement in the longitudinal girders and cross-girders.

With reference to the first mentioned, it would appear that members of unequal size would be dissimilarly influenced by temperature changes, causing thereby undue stress in the columns and floor system which would combine with the secondary stresses induced by the rigid connections of the upper and lower decks. The arch ribs, owing to their unequal size, would undoubtedly, when subjected to temperature changes, exert a like influence on the spandrel columns and supported deck, resulting in strains and cracks in the floor system and weaker members, though granting certain elastic deformation through-

* Portland, Ore.

Mr. Holmes.

out the system. The stress induced as above, combined with the possible variation in temperature between the arch ribs and lower chords, should, in the writer's opinion, merit consideration. The possibility of the dissimilar temperature influence is further emphasized by the fact that the lighter members are all on the south, or more exposed side of the structure. The effects of these stresses might be modified, but not eliminated, by constructing expansion joints at the intersection of the arch axis and the line of temperature thrust, or at the spandrel columns nearest this intersection.

It is gratifying to note the thorough and systematic methods followed in the arch analysis, and that the method of influence lines is coming into more general use, as it is certainly the only method that can be relied on for structures subjected to the severe concentrations imposed by modern traffic. Under the common loadings of several years ago, it was thought sufficient to consider the arch under two or three conditions of loading. This method, though still used to a great extent, is entirely inadequate, and, if applied, gives no assurance that the maximum stresses in the arch have been determined. Failures have been averted only by providing an excess of material; but, at the present day, when economy of design is considered a factor of major importance, a more exact method of loading must be adopted. Since the position of the live load producing maximum moment at any point will vary in arches of different proportions and form, modern practice makes it imperative, in the interests of safety and economy, that the more complete and exact method of analyzing an arch for a load of unity at each load point be used. Influence lines may then be drawn for both the upper and lower fibers, and the exact maximum values due to both vertical and horizontal loads, either uniform or concentrated, may be determined, as the influence lines will show clearly the extent and general position of the loads for maximum stress. Undoubtedly, the design of an elastic arch is one of the most delicate problems of structural engineering, but if the designer is sufficiently familiar with his work, it may be performed without great difficulty. It is only by applying rational and systematic methods, such as were used in the design of the structure under discussion, and combining the economical design evolved with faultless construction, that the field of usefulness of this esthetic form of structure will become more generally recognized.

The author has used the method of constructing influence lines for moment and thrust. By constructing these lines for stress in extreme fiber, a great deal of time may be saved by applying a modification of the stress equation for homogeneous beams,

$$S = \frac{M}{\frac{I}{c}},$$

Mr. Holmes.

where S = stress in extreme fiber,
M = maximum sectional bending moment,
I = moment of inertia,
and c = distance to neutral axis.

When the fiber stress is wholly compression, and considering the additional factors,

r = least radius of gyration,
N = thrust normal to the section,
A = area of transformed section,
I' = moment of inertia of transformed section,
and e = eccentricity,

the unit stress in the concrete will be $\frac{N}{A}$, and the unit flexural will be $\frac{Mc}{I'}$. Considering the combined section, we have

$$(M\,c\,A) + (N\,I') = S\,A\,I',$$

$$\text{or } S = \frac{N}{A} + \frac{M\,c}{I'}.$$

Equating $A\,r^2 = I'$, and $M = N\,e$, we have

$$S = \frac{M'\,c}{I'}, \text{ where } M' = N\left(e + \frac{r^2}{c}\right).$$

Computing the values of $\frac{r^2}{c}$ for the various sections, and considering this distance as the moment arm, and the fiber stress as varying as the moment, an influence line drawn for the moments thus computed will serve directly as an influence line for fiber stress. The writer has found that this direct method eliminates many chances for error, and saves considerable time. Of course, when the tensile stress in the concrete is ignored, as it should be, and the resultant stresses are tensile, it will be necessary to plat the influence lines for moment and thrust, and compute the stress in extreme fiber in the usual manner.

In connection with the discussion of stress in an arch of this type, a most important feature is the method of construction. The writer believes that cracks and initial stresses, as often developed, may be traced directly to the construction methods, and a description of the latter would complete this valuable paper.

With reference to the curved tension reinforcement in the longitudinal and cross-girders, it may not be unreasonable to ask why the main tension reinforcement was not placed in a horizontal position, and lighter and shorter lengths of steel bars used to reinforce the haunches. It appears to the writer that the method used

Mr. Holmes.

does not accomplish the results in the most economical way. Although the distribution of metal throughout the girders is ideal, it would appear that the same results could be accomplished with a smaller quantity of steel.

As shown by the following demonstration (see Fig. 29):

Let P = total stress in bar;
p = normal pressure per linear unit;
ds = single element.

Then the vertical component $= p\,ds$; $\cos.\,\theta = pr\,d\,\theta$; $\cos.\,\theta = X$.

$$\Sigma\,X = \int_c^B pr,\cos.\,\theta,\,d\,\theta + c = pr\,\sin.\,\theta + c.$$

$\Sigma\,X = 0$ when $\theta = 0$, therefore $c = 0$.

$$\Sigma\,X = pr\,\sin.\,\theta \quad \ldots\ldots\ldots\

Mr. Holmes.

On the other hand, it would not be economical to compensate for the effect of the vertical components of force by increasing the sectional area of metal at the critical section.

The distribution of the summation of the vertical forces referred to would follow the sine curve and result in considerable increase of moment at the center of the girder.

The painstaking care exercised by the designers in the close attention to details is to be commended, as there is very little to be guessed at by the prospective builder in figuring his estimates of costs.

Another important feature, in the writer's opinion, was the exclusion of piles in which fresh or unset concrete is placed against the soil. Considerable experience in connection with concrete piles under various subsoil conditions has demonstrated conclusively, to the writer's satisfaction, that reliable results in this most important feature of foundation work may be expected only by using piles in which the unset concrete is not allowed to come in contact with the subsoil. The many failures resulting from the use of concrete piles constructed in immediate contact with the soil should fully justify the statement that those of the latter type are not to be recommended where permanent and reliable results are desired.

The writer is pleased to note that the conventional sand cushion under the wood block pavement has been omitted. Considerable experience on the writer's part in constructing wood block pavements on bridge floors and reconstructing the floor systems of various bridges has demonstrated that the blocks are bound to heave and buckle, due to the shifting of the sand cushion, particularly when laid on a grade. The sand cushion will also give trouble where the blocks are laid with open joints, owing to the possibility of water leaching through the joints and displacing the cushion. The method used by the author has many advantages, and effectively water-proofs the bottom of the blocks.

Owing to the great variety of traffic that will no doubt be carried by this viaduct, it will be interesting to note the results obtained, under various climatic conditions, by constructing a wood block pavement on a grade of 5.5% and using the ¾-in. lath separators between the rows. This method has been tried before, but it would seem that observation of the great variety of types of street users that may be expected to pass over this structure will result in interesting and valuable data.

In reviewing the paper, one cannot help being most favorably impressed with the fact that, in drafting the specifications, an effort was made to eliminate as far as possible the uncertain characterization of work "satisfactory to the engineer", to make perfectly clear to the prospective builder just what was to be required, and to express the views of the engineer in advance. The writer does not wish it

Mr. Holmes.

to be inferred, however, that the engineer should have no discretion whatever in regard to carrying out the contract, but it may be unhesitatingly stated that this feature is one of the most important matters before the Engineering Profession to-day. The "satisfaction" of an engineer is a factor over which the contractor has no control, and it would be possible for a contractor to perform work absolutely above criticism, and yet not be able to compel the engineer to be "satisfied". Most contractors take pride in their work, and, as a matter of business policy and to maintain a reputation as a reliable contractor, make conscientious efforts to perform their work to the satisfaction of the engineer. Reliable contractors realize that competent supervision on the part of the engineer is absolutely essential. The making of a contract to the effect that work shall be performed to the "satisfaction of the engineer" and the letting of contracts without advance engineering information have been responsible for considerable pooling and combination in the contract business. The very idea of contracting to perform certain work is that such work can be planned and estimated in advance, as a result of experience with similar work, under like conditions, so that the contractors may have full knowledge of what is to be required and be able to fix with reasonable accuracy the cost of performing it. Contract work should be confined to that which involves quantities and conditions which may with reasonable certainty be determined in advance by experienced engineers. Work which cannot be determined in this way should be let on a basis of reasonable cost, plus a just percentage for profit. It should be clearly understood, however, in this latter case, just what items should be included in the cost.

In defense of the Profession, however, it should be considered that many of the weaknesses of specifications governing, or supposed to govern, public work may not be the fault of the engineer, as it is undoubtedly true that many of the absurd and unjust requirements, which are injurious both to contractors and to the public, are provided by law and by city charters. That many of the clauses found in public work specifications are not necessary is shown by the faithful and satisfactory completion, by contractors all over the country, of contracts with business firms where no such clauses are considered necessary, and where there exists recognition of the relation between the responsibility of the contractor and the requirements for the bond and retained percentage.

One of the main reasons that public work is often more expensive and less satisfactory than work contracted for by private firms is the fact that most of our laws compel the award to the lowest bidder. This, combined with the many other uncertain requirements, has in many cases proved most injurious to the public welfare.

Mr. Holmes.

It is quite generally supposed that profits from public work contracts are so great that anything that can be imposed on the contractor is so much clear gain. Needless risk should not be imposed on a contractor. Any one possessing an elemental knowledge of business principles should know that no competent and financially responsible contractor will take a gambling risk without charging enough on his bid to save himself from loss. Of course, there are contractors who will bid low in order to obtain the work, but, in the majority of cases, they are either financially irresponsible, or shrewd enough to make up their loss by slighting the work. Work let to the contractor of this latter class is bound to cost in the long run a good deal more than had it been let to a competent and reliable man who understands his business.

The writer has had considerable experience in contract work, both public and private, and has learned to appreciate the integrity and desire of contractors to perform work creditable to their reputation. The Profession should stand as a unit in making an effort to overrule the many unfair clauses of almost universal use in specifications, as these clauses have their part in limiting competition, in lowering the standard of honesty among contractors, and in lessening the dignity and standing of the Profession. By cultivating the confidence of reputable contractors and providing for competitive bidding on public contracts, immense sums may be saved to the taxpayers. A continuation and more wide-spread adoption of the policy followed by the engineers in connection with the contract for the structure under discussion will accomplish the desired results. The author deserves the thanks of every member of the Profession for his effort.

Mr. Upson.

M. M. Upson,* Assoc. M. Am. Soc. C. E. (by letter).—As is usual in a descriptive paper of this character, limited space has prevented Mr. Howard from taking up in detail the reasons for selecting the component parts of the structure. In particular, a short account of the selection of Raymond piles by the contractor, and their acceptance by the engineers, will show the care and forethought which characterized this entire work, and can now be seen to have saved both time and money.

Up to 1912, with one exception, the only concrete piles used in Kansas City were pre-cast. Mr. Howard had used pre-cast piles on a somewhat similar viaduct within half a mile of the present structure. It was natural, therefore, that the specifications should allow pre-cast as well as cast-in-place piles, the latter being limited to those in which "fresh or unset concrete is not placed against the soil." Pre-cast piles 25 ft. long were estimated as being necessary, it being provided that a variation in length could be ordered if required when the

* New York City.

Mr. Upson.

work began, and it was specified that such piles were to be jetted into place.

The soil of the site is a typical Missouri River fill overlying a very fine and comparatively clean sand, admirably suited to jetting, and into which the contractor would have experienced no difficulty in placing the 25-ft. pre-cast pile. On the east end of the viaduct, where it approaches the rock cliff, erosion subsequent to the initial placing of the soil had evidently taken place, as the sand and rock were covered with a very soft muddy material which necessitated piles at least 30 ft. long.

It is apparent that had pre-cast piles been used, about 10% of their total number would have been increased fully to 30 ft. in length on this account. The remainder, by the use of a jet, would have penetrated 25 ft., so that the 2 610 piles would have totaled 66 555 lin. ft.

After a careful study of the conditions, Raymond piles were selected. They were driven without jetting to a sufficient penetration to support safely the maximum load of 35 tons each. By driving to an agreed penetration to the last inch, a uniform bearing value was obtained, the lengths varying to meet the different soil conditions. They ranged from 12 to 30 ft., and in some individual piers there was a length variation of as much as 5 or 6 ft. By the use of the Raymond system, it was unnecessary to predetermine the lengths of the piles, and the city paid for only the actual number of feet driven. In this instance, the 2 610 piles totaled 46 625 lin. ft., which showed a saving of 19 930 ft. over the prescribed 66 550 lin. ft. of pre-cast piles. At the contract price, $1.20 per ft., this saved the city about $24 000, or 43% of the total cost of the piling.

It may be well to point out one or two reasons why, in the writer's opinion, a less length of pile may safely carry the same load. In this instance, as in most pile installations, the piling element does not penetrate to rock or hardpan, but carries its load by surface friction. The friction per square foot of surface in a given soil is dependent on several factors:

1.—The angle between the soil and the surface. It is obvious that the friction on a vertical surface can be no more than the shear of the adjacent soil, though the friction on a perfectly horizontal surface will be equivalent to the compressive strength of the soil. The vertical surface friction varies between these two limits. On that account a tapered friction pile has a great advantage over the straight cylindrical pile. The writer has frequently tested tapered piles to a surface friction of more than 2 000 lb. per sq. ft. without rupture.

2.—The degree of compression of the soil which is attained by the driving. This depends on the amount of abuse to which the driven

pile or form may be subjected. The necessary cushion to protect the concrete of a pre-cast pile limits its effectiveness in this function. Mr. Upson.

3.—The maintaining of the compression after it has been attained through the driving. This is effectively accomplished in the pre-cast pile, as it is also in the Raymond type, with its spirally reinforced and corrugated shell, which is left in the ground.

In addition to the saving of cost by reason of the shorter lengths required, the use of Raymond piles avoided a difficulty which would undoubtedly have proved both costly and troublesome to the contractor. This is the operation of jetting on the site.

The work was in a congested district, with unusually heavy street and railroad traffic. Had pre-cast piles been used, the disposal of the water from the jets would have been a great problem. A filter of some kind would have been required to separate the sand from the water, as the sewers in that part of Kansas City have a very small fall, which would prevent them from carrying water containing any large quantity of sand.

The jetting of piles in a city always entails many difficulties, especially when fine and light soil, easily carried in suspension, is encountered. It is, of course, quite impossible to carry the water on the surface, because of streets and crossings which occur at close intervals. It will be seen, therefore, that, by avoiding this difficulty, the contractor saved himself much annoyance and expense.

Time is another factor of economy affected by the use of Raymond concrete piles on this work. Had pre-cast piles been used, most of them would have been cast during the winter. A minimum time of 30 days was specified, and under ordinary winter conditions this time would have been doubled before the piles had cured sufficiently to be safely handled. As a matter of fact, the weather during this work was unusually mild, so that the temperature delay would not have been serious. However, the necessary 30 days after the receipt of the reinforcing rods, which usually requires several weeks for delivery on the ground, necessitates an interval of from 6 to 8 weeks between the signing of the contract and the placing of the footings. Furthermore, a slight variation in the underlying soil conditions may involve the contractor in a further delay of from 30 to 60 days, in order to provide longer piles to meet the varied requirements.

The actual money saving by cutting down the time is frequently lost sight of by the contractor as well as by the owner. After a construction project is determined on, and the preparations are made, the money loss to the owner usually can be computed accurately, and often amounts to several hundred dollars a day, and all wise contractors readily realize the fact that their profit, under ordinary conditions, is inversely proportional to the time required to complete the work. Sev-

Mr. Upson.

eral instances have come to the writer's attention where the loss to the owner and a proportional amount to the contractor, on account of the delay incident to choice of improper foundation methods, has involved a sum greater than the total cost of the foundation itself.

To summarize: Raymond piles were selected, first, because of the saving in length, which, in this instance, amounted to 43%, or $24 000, on the total expenditure of $56 000; second, the saving of time, which may be estimated at not less than 30 days, as the footings and superstructure followed very closely on the driving of the piles; third, the avoidance of annoying and expensive pumping equipment and methods for taking care of waste water incident to the jetting.

Mr. Slettum.

E. A. SLETTUM, ESQ.* (by letter).—The investigation of the columns for the Twelfth Street Trafficway Viaduct was made by the writer, then in the employ of Waddell and Harrington, the designers of the structure, as Assistant Engineer. On account of the great number of columns and their size, it was of the greatest importance, in order to obtain an economical and safe structure, to determine, with as great precision as practicable, the various bending moments that might occur. The most difficult problem was the determination of

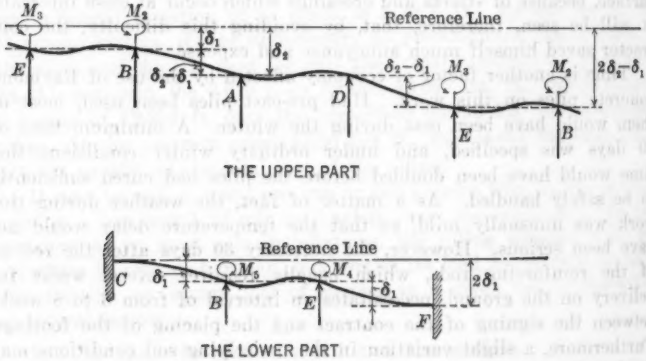

FIG. 29.

bending moments due to wind loads. A typical bent, as shown by Fig. 6, was chosen, and the problem was solved, as mentioned, by breaking the frame up into two parts, one upper part, *ABED*, and one lower part, *BCFE*, the cross-girder, *BE*, being common for both. The two parts were then straightened out, as shown in Fig. 29, and calcu-

* Christiania, Norway.

lated as continuous beams on supports at different levels, the internal moments, M_2 and M_3, of the lower part acting as external moments on the upper part, and, *vice versa*, the internal moments, M_4 and M_5, of the upper part acting as external moments on the lower part. Mr. Slettum.

The frame was assumed to be rigidly fixed at the bases, C and F. All members of the frame, except the cross-girder, BE, had variable cross-sections, and, therefore, the three-moment equation for beams with variable moments of inertia was used. This equation, not being published in any book known to the writer, was developed by him in connection with this work, and may possibly be of some interest.

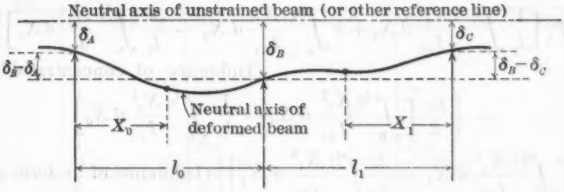

FIG. 30.

Let ABC be any three consecutive supports in a continuous beam;

I_{X0} be the general value of the moment of inertia in the left-hand panel (Panel AB);

I_{X1} be the general value of the moment of inertia in the right-hand panel (Panel BC);

X_0 be the distance from the left-hand support to any point in the left-hand panel;

X_1 be the distance from the right-hand support to any point in the right-hand panel;

P_0, p_0, and M_0, respectively, be any concentrated load, uniform load per unit, and external bending moment, in the left-hand panel.

P_1, p_1, and M_1, respectively, be any concentrated load, uniform load per unit, and external bending moment, in the right-hand panel;

kl_0 and kl_1 be the distances, respectively, from the left-hand support in the left-hand panel and from the right-hand support in the right-hand panel, to any concentrated load or external bending moment;

E be the modulus of elasticity of the material; and

M_A, M_B, and M_C be the internal moments (continuity-moments) at A, B, and C, respectively.

Mr. Slettum.

Then, with the notation shown in Fig. 30, the three-moment equation for beams with variable moments of inertia is written thus:

$$M_A\left[\frac{1}{l_0}\int_0^{l_0}\frac{X_0}{I_{X0}}\,dX_0-\frac{1}{l_0^2}\int_0^{l_0}\frac{X_0^2}{I_{X0}}\,dX_0\right]$$
$$+M_B\left[\frac{1}{l_0^2}\int_0^{l_0}\frac{X_0^2}{I_{X0}}\,dX_0+\frac{1}{l_1^2}\int_0^{l_1}\frac{X_1^2}{I_{X1}}\,dX_1\right]$$
$$+M_C\left[\frac{1}{l_1}\int_0^{l_1}\frac{X_1}{I_{X1}}\,dX_1-\frac{1}{l_1^2}\int_0^{l_1}\frac{X_1^2}{I_{X1}}\,dX_1\right]$$
$$=-\Sigma\left\{P_0\left[\frac{1}{l_0}\int_0^{k\cdot l_0}\frac{X_0^2}{I_{X0}}\,dX_0+k\int_{k\cdot l_0}^{l_0}\frac{X_0}{I_{X0}}\,dX_0-\frac{k}{l_0}\int_0^{l_0}\frac{X_0^2}{I_{X0}}\,dX_0\right]\right.$$
$$\left.+P_1\left[\frac{1}{l_1}\int_0^{k\cdot l_1}\frac{X_1^2}{I_{X1}}\,dX_1+k\int_{k\cdot l_1}^{l_1}\frac{X_1}{I_{X1}}\,dX_1-\frac{k}{l_1}\int_0^{l_1}\frac{X_1^2}{I_{X1}}\,dX_1\right]\right\}$$

(Influence of concentrated loads)

$$-\left\{\frac{p_0}{2}\left[\int_0^{l_0}\frac{X_0^2}{I_{X0}}\,dX_0-\frac{1}{l_0}\int_0^{l_0}\frac{X_0^3}{I_{X0}}\,dX_0\right]\right.$$
$$\left.+\frac{p_1}{2}\left[\int_0^{l_1}\frac{X_1^2}{I_{X1}}\,dX_1-\frac{1}{l_1}\int_0^{l_1}\frac{X_1^3}{I_{X1}}\,dX_1\right]\right\}$$

(Influence of uniform loads)

$$-\Sigma\left\{M_0\left[\frac{1}{l_0^2}\int_0^{l_0}\frac{X_0^2}{I_{X0}}\,dX_0-\frac{1}{l_0}\int_{k.l_0}^{l_0}\frac{X_0}{I_{X0}}\,dX_0\right]\right.$$
$$\left.+M_1\left[\frac{1}{l_1^2}\int_0^{l_1}\frac{X_1^2}{I_{X1}}\,dX_1-\frac{1}{l_1}\int_{k.l_1}^{l_1}\frac{X_1}{I_{X1}}\,dX_1\right]\right\}$$

(Influence of exterior bending moments or pairs of forces)

$$+E\left[\frac{\delta_B-\delta_A}{l_0}+\frac{\delta_B-\delta_C}{l_1}\right]$$

(Influence of yielding supports, or supports at different levels).

In this equation, P and p are introduced with positive sign when producing tension on the under side of a beam freely supported on two supports, and M is introduced with positive sign when producing tension on the under side of the beam between the left-hand support and the point of application in the left-hand span (that is, when acting counter clockwise), and between the right-hand support and the point of application in the right-hand span (that is, when acting clockwise).

The work was somewhat simplified in this case by having no other exterior loads with which to deal, other than the bending moments at B and E. The quantities, $\int\frac{X}{I}\,dX$, $\int\frac{X^2}{I}\,dX$, etc., were determined graphically in the manner indicated by Fig. 5.

On the basis of these calculations, the writer has devised a more general application of the theory of the continuous beam to problems of this nature, which is set forth in the following discussion.

Mr. Slettum.

He does not believe that the method will be used to any great extent, except for rather simple frames, the labor of solving the many equations being enormous, on account of the calculations having to be carried through with the most rigid accuracy and to the last decimal point, in order to obtain correct results, the slightest rounding off resulting in an error which increases rapidly with the number of equations.

FIG. 31.

For problems, such as the case in hand, having not more than eight or ten statically indeterminate quantities, the writer has used the method with entire success, although even such simple cases require careful calculations.

The Application of the Theory of the Continuous Beam in Calculating the Stresses in Frames with Rigid Connections. A frame with rigid connections is one in which all members are connected at

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their intersection points in such a way that the angle between any two members meeting at a point will remain constant under all conditions of loading. A change in the direction of the end tangent of one member, therefore, will cause the same change in the direction of the end tangents of all other members meeting at the point. As this is the basis of the theory of the ordinary continuous beam, it is self-evident that this theory is applicable to such frameworks as described herein.

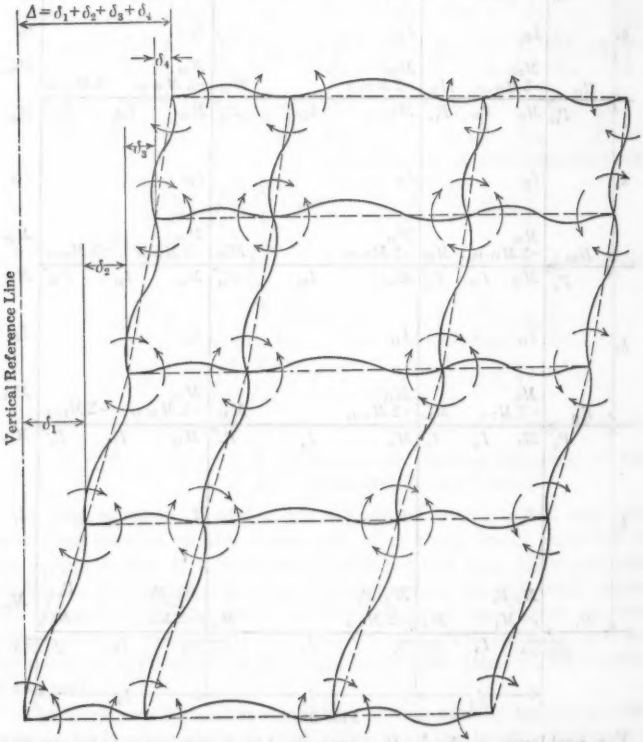

FIG. 32.

To illustrate the general application of the theory of the continuous beam for calculating the stresses in such frames, take the framing bent of a four-story structure, as shown in Fig. 31.

The moments of inertia have been assumed to be different for different members, but constant throughout the length of each separate member. The only difference in the case of a variable moment of

inertia is that the coefficients in the three-moment equation change as shown under the theory of the continuous beam. Mr. Slettum.

The frame will be assumed to be loaded in any manner, horizontally or vertically, by forces acting in its plane.

The influence of axial stresses on the deformation of the frame will be neglected.

$H_I + H_{II} + H_{III} + H_{IV} + H_V$ is the resultant of all horizontal forces, including wind load, concentrated at the different floor levels.

Under the application of external forces, the frame will deflect from its original position. The direction in which it will deflect cannot always be told beforehand. It will be convenient to assume,

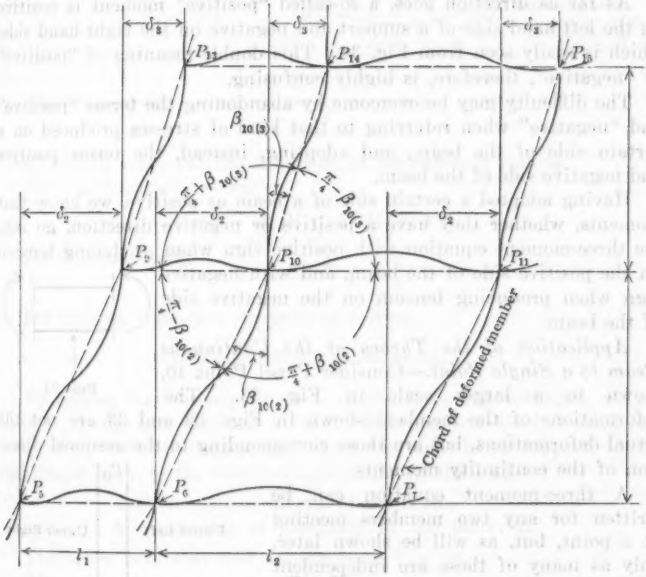

FIG. 33.

therefore, that the frame always deflects in a certain fixed positive direction. By the sign of the figure giving its value we can then tell the actual direction. If it is plus, the frame deflects in the direction assumed; if it is minus, it deflects in the opposite direction. This also applies to the unknown continuity moments.

Assume that from left to right is the positive direction for deflections.

Assume a clockwise direction as positive for moments when applied to the panel points, and counter clockwise when applied to the members.

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In Fig. 32 is shown the deflected frame with the moments applied to the panel points. As each panel point is in equilibrium, the algebraic sum of the moments about any point must equal zero; hence, if all moments but one about a point are assumed to act in a positive direction, this latter moment will equal the sum of the others, and will be negative, as shown in Fig. 32.

When speaking about beams, it is the usual practice to term moments producing tension on the bottom side "positive", and those producing tension on the top side "negative", the reason being that they go into the three-moment equations with positive and negative signs, respectively.

As far as direction goes, a so-called "positive" moment is positive on the left-hand side of a support and negative on the right-hand side, which is easily seen from Fig. 34. This double meaning of "positive" or "negative", therefore, is highly confusing.

The difficulty may be overcome by abandoning the terms "positive" and "negative" when referring to that kind of stresses produced on a certain side of the beam, and adopting, instead, the terms positive and negative side of the beam.

Having adopted a certain side of a beam as positive, we know that moments, whether they have a positive or negative direction, go into the three-moment equation with positive sign when producing tension on the positive side of the beam, and with negative sign when producing tension on the negative side of the beam.

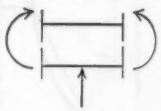

FIG. 34.

Application of the Theory of the Continuous Beam to a Single Point.—Consider Panel Point 10, shown to a larger scale in Fig. 33. The deformations of the members shown in Figs. 32 and 33 are not the actual deformations, but are those corresponding to the assumed direction of the continuity moments.

A three-moment equation can be written for any two members meeting at a point, but, as will be shown later, only as many of these are independent of each other as there are members meeting at the point, less one. Thus, if there are four members meeting at a point, we can write only $4 - 1 = 3$ independent equations.

For convenience, therefore, we shall confine ourselves to members forming either a lower left, upper right, or lower right, corner, as shown in Fig. 35.

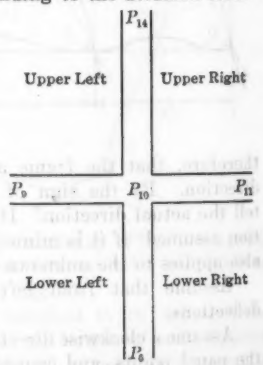

FIG. 35.

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The three-moment equation for the ordinary continuous beam is:

$$M_0 \frac{l_0}{I_0} + 2 M_1 \left(\frac{l_0}{I_0} + \frac{l_1}{I_1} \right) + M_2 \frac{l_1}{I_1}$$

$$= C_0 + C_1 + 6 E \left(\frac{\delta_1 - \delta_0}{l_0} + \frac{\delta_1 - \delta_2}{l_1} \right) \ldots\ldots\ldots (1)$$

The subscript, 0, refers to the left-hand span, and the subscript 1, to the right-hand span. (See Fig. 36.)

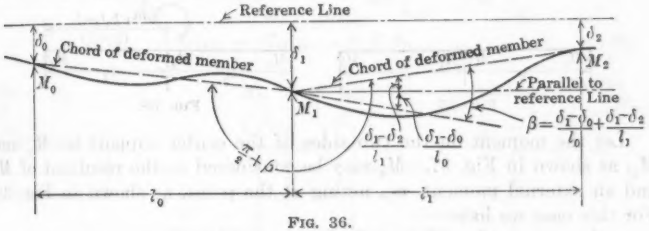

FIG. 36.

C_0 and C_1 represent the influence of the external loading.

For concentrations we have:

$$C = - \Sigma P \frac{l}{I} k l (1 - k^2).$$

For uniform loads:

$$C = - \frac{1}{4} p l^2 \frac{l}{I}.$$

For external moments (or pairs of forces):

$$C = - \Sigma m \frac{l}{I} (3 k^2 - 1).$$

By adding the subscript, 0 or 1, in the foregoing formulas, we get C_0 or C_1, as the case may be. $k l$ is the distance from the left-hand support, in the left-hand span, and from the right-hand support, in the right-hand span, to the point of application of the load.

I = moment of inertia of the beam.

E = modulus of elasticity of the material.

The other notations will appear from Fig. 36.

The external moments, m, go into the equation with positive sign when acting counter clockwise in the left-hand span and clockwise in the right-hand span.

The unknown continuity moments, M_0, M_1, and M_2, are assumed to act in such a direction that they produce tension on the positive side of the beam, which is the under side.

From Fig. 36 it is seen that the angle, β, which represents the change in angle between the two members, equals:

$$\beta = \frac{\delta_1 - \delta_0}{l_0} + \frac{\delta_1 - \delta_2}{l_1} \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

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Therefore, the three-moment equation (1) can be written:

$$M_0 \frac{l_0}{I_0} + 2\, M_1 \left(\frac{l_0}{I_0} + \frac{l_1}{I_1}\right) + M_2 \frac{l_1}{I_1} = C_0 + C_1 + 6\, E\, \beta \ldots\ldots (3)$$

Now, a frame differs from an ordinary continuous beam therein, that, instead of having the same moment, M_1, on both sides of the center support, it has different moments on the two sides, due to the action of the other members meeting at the point.

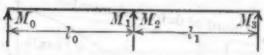

FIG. 37.

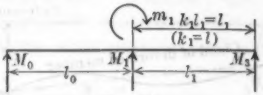

FIG. 38.

Let the moment on the two sides of the center support be M_1 and M_2, as shown in Fig. 37. M_2 may be considered as the resultant of M_1 and an external moment, m_1, acting at the point, as shown in Fig. 38. For this case we have:

$$M_0 \frac{l_0}{I_0} + 2\, M_1 \left(\frac{l_0}{I_0} + \frac{l_1}{I_1}\right) + M_3 \frac{l_1}{I_1} = -m_1 \frac{l_1}{I_1} (3\, k_1{}^2 - 1) + 6\, E\, \beta.$$

But $$k_1 = 1, \text{ and } m_1 \frac{l_1}{I_1} (3\, k_1{}^2 - 1) = 2\, m_1 \frac{l_1}{I_1}.$$

Moving this term to the left side we get:

$$M_0 \frac{l_0}{I_0} + 2\, M_1 \frac{l_0}{I_0} + 2\, (M_1 + m_1) \frac{l_1}{I_1} + M_3 \frac{l_1}{I_1} = 6\, E\, \beta.$$

But $M_1 + m_1 = M_2$, therefore we have, in general, for such cases:

$$M_0 \frac{l_0}{I_0} + 2\, M_1 \frac{l_0}{I_0} + 2\, M_2 \frac{l_1}{I_1} + M_3 \frac{l_1}{I_1} = C_0 + C_1 + 6\, E\, \beta \ldots\ldots (4)$$

This is the three-moment equation as applied to frameworks.

From Equation (2) we see that β is positive when the angle between the chords of the deformed members measured on the positive side of the beam is increased, and negative when the same angle is decreased.

Choosing the under side of $P_9\, P_{10}$ as the positive side, we have (see Fig. 39 and also Fig. 33):

For "lower left":

$$\Sigma\, M_{17\text{-}18} \frac{l_1}{I_{15}} + 2\, M_{20} \frac{l_1}{I_{15}} - 2\, M_{19} \frac{h_2}{I_{12}} + M_{11} \frac{h_2}{I_{12}} = C_{15\,(9)} + C_{12\,(6)} - 6\, E\, \beta_{10\,(2)} \ldots\ldots\ldots\ldots\ldots (5)$$

For "upper left":

$$\Sigma\, M_{17\text{-}18} \frac{l_1}{I_{15}} + 2\, M_{20} \frac{l_1}{I_{15}} - 2\, M_{21} \frac{h_3}{I_{19}} + M_{29} \frac{h_3}{I_{19}} = C_{15\,(9)} + C_{19\,(14)} - 6\, E\, \beta_{10\,(3)} \ldots\ldots\ldots\ldots\ldots (6)$$

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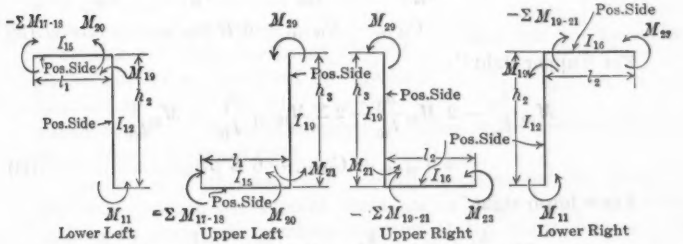
For "upper right":

$$M_{29}\frac{h_3}{I_{19}} - 2\,M_{21}\frac{h_3}{I_{19}} - 2\,\Sigma\,M_{19\text{-}21}\frac{l_2}{I_{16}} - M_{23}\frac{l_2}{I_{16}} = C_{19\,(14)} + C_{16\,(11)} - 6\,E\,\beta_{10\,(3)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

For "lower right":

$$M_{11}\frac{h_2}{I_{12}} - 2\,M_{19}\frac{h_2}{I_{12}} - 2\,\Sigma\,M_{19\text{-}21}\frac{l_2}{I_{16}} - M_{23}\frac{l_2}{I_{16}} = C_{12\,(6)} + C_{16\,(11)} - 6\,E\,\beta_{10\,(2)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$



Note that in these sketches the moments are shown applied to the members, and not to the panel points.

FIG. 39.

In these equations the first subscript for C refers to the member, and the second subscript, which is in parentheses, refers to the panel point for which C is to be figured. In the same way, the first subscript for β refers to the panel point, and the second subscript to the story of the frame. (See Fig. 33.)

From Fig. 40 it is seen that the positive side of the members in one panel is the inside, and in the next panel the outside, the inside of one panel being the outside of its neighbor. It would be necessary, therefore, to assume the positive side of one member and then go through the whole frame and mark the positive side, as shown in Fig. 40.

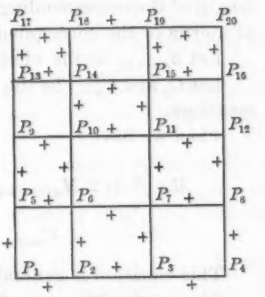


FIG. 40.

This would be very likely to cause some confusion, but the difficulty can be overcome simply by changing the signs throughout in the equations for "upper left" and "lower right". It is easily seen that this is equivalent to changing the positive side in "upper left" and "lower right", so that the inside of the panel to which the two members

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belong, in all cases, will be the positive side. The resulting equations for the point, P_{10}, therefore, are:

For "lower left":

$$\Sigma M_{17-18}\frac{l_1}{I_{15}} + 2M_{20}\frac{l_1}{I_{15}} - 2M_{19}\frac{h_2}{I_{12}} + M_{11}\frac{h_2}{I_{12}} = C_{15\,(9)} + C_{12\,(6)} - 6E\beta_{10\,(2)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

For "upper left":

$$-\Sigma M_{17-18}\frac{l_1}{I_{15}} - 2M_{20}\frac{l_1}{I_{15}} + 2M_{21}\frac{h_3}{I_{19}} - M_{29}\frac{h_3}{I_{19}} = -C_{15\,(9)} - C_{19\,(14)} + 6E\beta_{10\,(3)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

For "upper right":

$$M_{29}\frac{h_3}{I_{19}} - 2M_{21}\frac{h_3}{I_{10}} - 2\Sigma M_{19-21}\frac{l_2}{I_{16}} - M_{23}\frac{l_2}{I_{16}} = C_{19\,(14)} + C_{16\,(11)} - 6E\beta_{10\,(3)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

For "lower right":

$$-M_{11}\frac{h_2}{I_{12}} + 2M_{19}\frac{h_2}{I_{12}} + 2\Sigma M_{19-21}\frac{l_2}{I_{16}} + M_{23}\frac{l_2}{I_{16}} = -C_{12\,(6)} - C_{16\,(11)} + 6E\beta_{10\,(2)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (12)$$

As the rule of signs now is the same for all panels, the positive sign always being the inside of the panel, we can write the following general equation: Let P_n, P_{n+1}, and P_{n+2} be the consecutive supports from left to right of any two members of a frame forming a lower left, upper left, upper right, or lower right corner. Let M_n, $M_{n+1\text{ left}}$, $M_{n+1\text{ right}}$, and M_{n+2} be the corresponding moments. Let l_m and l_{m+1} be the length of the corresponding members. Let I_m and I_{m+1} be the moments of inertia of the corresponding members.

Let $\beta_{n+1\,(r)}$ be the change in angle between the two members.

Let C_n and C_{n+1} be the influence of external load on the corresponding members.

Then we have:

$$M_n\frac{l_m}{I_m} + 2M_{n+1\text{ left}}\frac{l_m}{I_m} + 2M_{n+1\text{ right}}\frac{l_{m+1}}{I_{m+1}} + M_{n+2}\frac{l_{m+1}}{I_{m+1}} = C_{m\,(n)} + C_{m+1\,(n+2)} + 6E\beta_{n+1\,(r)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

This equation is general, and the rule of signs is as follows:

The continuity moments, M_n, M_{n+1}, etc., enter with positive sign when producing tension on the positive side of the member, that

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is, the inside of the panel to which the member belongs. The quantities, C_m and C_{m+1}, enter with positive sign when the external loads act in such a way as to produce tension on the negative side of the member, that is, the outside of the panel to which the member belongs.

The quantity, $\beta_{n+1\,(r)}$, is positive when the angle between the two members, measured on the inside of the panel to which the two members belong, becomes greater; or, in short, follows the same rule as for the ordinary continuous beam.

To make the case absolutely clear, the positive sides of members and the sign for β have been shown in Fig. 41. The sign for β assumes that the frame is supposed to deflect in the positive direction, that is, from left toward right.

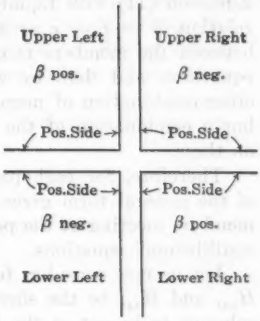

FIG. 41.

Number of Unknowns and Equations Required to Solve the Problem.—At each panel point there are as many unknown bending moments as there are members meeting at the point, less one, this latter being equal to the algebraic sum of the others and opposite in direction.

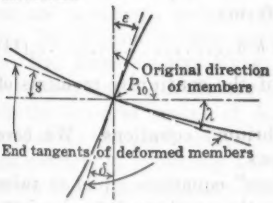

FIG. 42.

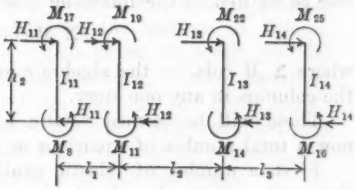

FIG. 43.

Besides, we have an unknown deflection, $\delta = \beta h$, for each story of the frame.

This gives a total number of unknowns equal to twice the number of members in the frame, less the number of panel points, plus the number of stories.

As already stated, we can write a three-moment equation for any combination of two members meeting at a point. However, only three —or in general the number of members meeting at a point, less one— are independent of each other.

To illustrate this, let us again consider the point, P_{10}. Equation (9) expresses the relation, $\delta = j$ (see Fig. 42); Equation (10) expresses the relation, $j = \epsilon$; and a combination of the two will express the relation, $\delta = j = \epsilon$.

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Equation (11) expresses the relation $\epsilon = \lambda$. A combination of Equation (11) with Equations (9) and (10), therefore, will express the relation, $\delta = j = \epsilon = \lambda$. Thus it is seen that the elastic relation between the members meeting at a point is fully expressed by three equations, and that, by writing the three-moment equation for any other combination of members, we write an equation which is nothing but a combination of the three equations, and accordingly dependent on these.

Therefore, for each panel point, we have a number of equations, of the general form given by Equation (13), equal to the number of members meeting at the point, less one. These will be termed "elastic equilibrium" equations.

Let us now consider, for example, the second story. Let H_{11}, H_{12}, H_{13}, and H_{14}, be the shears in the respective columns. Assume the columns to be cut at the floor levels, and the internal forces applied, as shown in Fig. 43.

As the system of forces is in equilibrium, we have:

$$\begin{aligned} M_8 + M_{11} + M_{14} + M_{16} + M_{17} + M_{19} + M_{22} + M_{25} \\ = h_2 (H_{11} + H_{12} + H_{13} + H_{14}) \\ = h_2 (H_V + H_{IV} + H_{III}) = h_2 \delta_{II}, \end{aligned}$$

where δ_{II} is the external shear in the story.

Similar equations can be written for each story. These equations can be written in the following general form:

$$\Sigma M \text{ cols.} = h \delta \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

where ΣM cols. = the algebraic sum of the continuity moments of the columns in any one story.

These will be termed "static equilibrium" equations. We have now a total number of equations as follows:

First, a number of "elastic equilibrium" equations equal to twice the number of members in the frame, less the number of panel points; and second, a number of "static equilibrium" equations equal to the number of stories in the frame. The total number of equations, therefore, equals twice the number of members in the frame, less the number of panel points, plus the number of stories, which is the number required to solve the problem.

Solution of the Problem.—Considerable time can be saved by following always a certain method in solving problems of this nature. The following is proposed:

(*a*) Make sketches of the structure, and show the quantities, as in Figs. 31 and 32. In Fig. 32 it is not necessary to show the deformed members themselves, but only their chords.

(*b*) Tabulate the values of l, h, I, $\frac{l}{I}$, $\frac{h}{I}$, and C.

(*c*) Write the equations with coefficients for the unknowns and the constants given by letters. For each story write the equation containing the term $6\,E\,\beta$ by itself, and in the other equations containing that term, substitute its value, as given by this equation. Arrange the unknowns in numerical order, and place all constants on the right-hand side of the equation. Mr. Slettum.

(*d*) Tabulate the numerical values of the coefficients for the unknowns and of the constants.

(*e*) Eliminate the unknowns.

(*f*) Find the unknowns by substitution.

The deflections are calculated from the general formula:

$$\delta = \beta h \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

where δ = deflection of any single story.

The total deflection of the frame is:

$$\Delta = \Sigma\,\delta = \Sigma\,\beta\,h \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

After the continuity moments have been calculated, each member may be assumed to be cut and freely supported at the panel points, and loaded with the external loads and the continuity moments. The further calculation is then as for an ordinary beam on two supports.

L. R. Ash,* M. Am. Soc. C. E. (by letter).—There are few cities in the United States where the freight-house and manufacturing districts are so sharply isolated from the business center as in Kansas City. In addition to this division between the sections of Kansas City, Mo., Kansas City, Kans., with a population of nearly 100 000 and its large and varied manufacturing interests, is also separated from the business center of Kansas City, Mo., by the bluffs which extend along the north and west sides of the city. The traffic between these districts is extremely heavy and, for a number of years, it has been seriously handicapped by bad grades and indirect routes. Mr. Ash.

The apparent need of a trafficway between Kansas City, Mo., and Kansas City, Kans., and to the West Bottoms, was great enough to enlist private capital in the enterprise of building the Sixth Street or Inter-city Viaduct some 8 or 10 years ago. This structure was built as a toll viaduct, and although it has proved to be a failure, from a financial standpoint, this failure in no way indicates that such a structure was not needed, but demonstrates the reluctance of the American public to pay tolls. During the times that the Inter-city Viaduct was open for traffic without charge, the structure was crowded with travel, both to and from Kansas City, Kans., as well as to and from the West Bottoms.

The question of building a viaduct at 12th Street, connecting with the West Bottoms, has been discussed for the last 15 years, and

* Kansas City, Mo.

Mr. Ash. although every one realized its great need, it seemed almost impossible to harmonize the various interests which in one way or another were affected by the building of the structure. The old viaduct at 12th Street provided only for street-railway traffic. All vehicular traffic between the West Bottoms and Kansas City, Kans., and the business center of Kansas City, Mo., had no thoroughfare from 6th Street on the north to 24th Street on the south, a distance of more than 1½ miles. When, in addition to this, we consider the fact that neither Sixth Street nor 24th Street led in the direction of the terminus for this large traffic, we see more clearly the great need of a trafficway leading to the heart of the business portion of the city.

Numerous meetings were held with representatives of the various organizations, such as the Team Holders Association, Real Estate Board, Commercial Club, etc., in an attempt to determine the most practicable structure, both as regards the requirements of traffic and the elimination of damages. These discussions were frequently very acrimonious, and politics cut no small figure in the handling of the proposition. The management of the whole enterprise was a shining example of the need of a more responsible and centralized authority in municipal affairs.

One of the great problems which confronted the promoters of the 12th Street Viaduct was that of grades. About 5% seemed to be the only practicable grade on 12th Street, and, in the minds of a great many people, this grade was prohibitive when heavy vehicular traffic was considered. The grade, as originally established, was 5.09%, and this would have necessitated a cut of about 17 ft. at the top of the hill, which would have resulted in a very heavy property damage estimated at about $250 000. This matter came up while the writer was City Engineer of Kansas City, Mo., and he suggested the elimination of this property damage by raising the grade to within about 4 ft. of the established grade at Washington Street. This increased the viaduct grade to 5.52%, a very slight change from the old one; and the opposition to the structure, thus avoided, abundantly justified the change.

The advent of the motor truck has decreased the necessity for lower grades, except where reasonably attainable, but, as a concession to the demand for a low-grade trafficway between the Bottoms and the business section, a lower deck was provided on the viaduct with a maximum grade of about 2½ per cent. It is expected to utilize this portion of the viaduct by building roadways along the bluff leading to the north and to the south from the east end of this lower deck. In the writer's opinion, the building of these roadways will not be justified, as they will lead to the old trafficway at 6th Street on the north, or to 17th Street on the south, neither of which points is in the line of travel, and from which there will be bad grades and other

conditions not inviting to heavy vehicular traffic. It would seem that a better solution of the problem would be to tunnel from the east end of the lower deck to about 14th and Wyandotte Streets, thus insuring a very easy grade and a direct route for the distribution of the traffic from the West Bottoms. Mr. Ash.

Now that the structure has been completed and thrown open to the public, the great need of it is proved by the very great traffic, both vehicular and by street cars, using it; and this is drawn to the structure in spite of the fact that the approaches to the viaduct have not been paved, as they await the settlement of the embankments at each end. The structure saves several minutes to each of the many people who go each day from the eastern portion of the city to the West Bottoms, and the 5.52% grade has proved to be no serious handicap, either to motor or horse-drawn vehicles.

It is interesting to note that the building of the viaduct has resulted in enhancing real estate values and rentals on East 12th Street and the adjacent streets for a distance of 3 or 4 miles eastward from the structure. This is because the opening of 12th Street provides a direct way to the West Bottoms for the large number of people employed there, without having to transfer or travel several blocks out of their way. The 12th Street cars, although greatly increased in number, are crowded with passengers, and the building of the viaduct has fixed 12th Street as the principal east and west artery of the city.

Mr. Howard has gone very fully into the design and construction features, and there is no additional comment to be made except to remark on the very satisfactory result, as shown by the finished structure. The surface finish of the concrete is not all that could be desired, but it is believed that it is in keeping with the character of the structure and, when conditions in the neighborhood are considered, it is thought that the more expensive methods of treating concrete surfaces would not have been justified in this case.

The specifications as drawn would permit the engineer to require the contractor to put a cement-gun surface on all columns and cantilevers, as well as on the outside surfaces of the longitudinal girders, but, in the writer's opinion, the attempt to place this finish on surfaces of the character in this structure, would not result satisfactorily, and it is believed that the surface as produced by the forms, with the rough places smoothed off with hammer and chisel and in certain places filled with mortar, has been the best treatment practicable.

The treatment of concrete surfaces is receiving a great deal of attention, and various methods have been advanced for producing a surface which is smooth and, at the same time, does not present the expressionless effect of a smooth mortar surface. The writer believes

Mr. Ash. that specifications can most easily provide for the finish of concrete surfaces by requiring the bidders to tender on two or three different kinds of treatment at a stated price per unit of surface, leaving the engineer to choose that method which, in his opinion, most nearly suits the condition when the forms are removed. Of course, all specifications should require forms to be built of smooth lumber, with tight joints and true to line.

The character of surface desired should be determined by the general characteristics of the structure, its location, type of construction, etc. No attempt should be made to give the concrete the appearance of stone or other material, and it is thought that, with a few exceptions, a reasonably smooth surface which is produced by good forms and well-spaded concrete, is better than can be secured by special treatments.

On the 12th Street Viaduct 22 ft. of the upper deck is set aside for street-car tracks, and a roadway, 30 ft. wide on the south side, is left for vehicles, a sidewalk 5 ft. wide extending along the south side of the structure. Although this arrangement has some advantages over that of placing the street railway in the center of the roadway, it would seem that the space between the hand-rails would be used more efficiently with the tracks in the center, as is the case in an ordinary city street. About the only justification for placing the tracks in a space independent of the roadway, is the length of the structure and the grade. It is believed that a sidewalk, 8 ft. wide, and a clear roadway of about 50 ft. between the curbs, with street-railway tracks in the center, would have resulted in a structure which would have cost somewhat less and would have been better suited to the general traffic conditions. As it is, vehicles on the north side of 12th Street, at each end of the viaduct, must cross the street railway tracks.

The use of wood blocks for paving the roadway portion has proved entirely satisfactory. It was feared that the grade would cause the pavement to be slippery, but the blocks were laid with a lath between each row, and the remaining space was well filled with broken stone. The blocks are 3 in. wide, and the resulting surface undoubtedly provides a better footing for horses than stone block, asphalt, or brick. The writer has observed heavily loaded teams turning out of the street railway portions of certain streets where there is stone block pavement, to the wood block pavement at the sides, because of the better footing secured on the latter.

The laying of the wood blocks on the viaduct without a sand cushion has been entirely satisfactory, and demonstrates the wisdom of omitting one of the great sources of trouble with pavements of this type. With care in finishing the concrete base, the surface can

be brought to true crown and grade without serious trouble, and when this is done there is no justification whatever for the use of a sand cushion. Mr. Ash.

The structure represents a great amount of careful study of the architectural treatment as well as the engineering features, and it is becoming more and more apparent that engineers should give consideration to matters pertaining to the appearance of structures of this character. There seems to be little excuse for absolutely neglecting the esthetic treatment of engineering structures, and although the Profession has been prone to do this, it is gratifying to note a marked change in its attitude in this matter.

JOHN LYLE HARRINGTON,* M. AM. SOC. C. E. (by letter).—So much of the Twelfth Street Trafficway Viaduct is of usual construction, and the author has exposed so fully the elements which are of interest to the engineer who is thoroughly experienced in the design of reinforced-concrete structures, that few matters pertaining to the viaduct require further comment. Mr. Harrington.

Some designers of reinforced-concrete structures consider the work so simple that their plans consist of a few general drawings showing principal dimensions, the number, size, and approximate position of reinforcing bars, and with a few notes pertaining to ties, constituents of concrete, and character of finish; and their specifications consist of little more than references to standard specifications for materials, a brief description of the structure, and the clauses relating to bids. Those of another class, generally of European training, go to the opposite extreme, and make extremely full plans and all the detailed calculations necessary for the exact determination of the stresses in continuously-framed structures, composed of uniformly perfect materials, moulded and placed exactly in accord with the plans. Both are in error. Ill-considered, carelessly-made plans and loosely-drawn specifications are responsible for a large number of defective structures and for many costly failures. Lax supervision is almost sure to follow such designing; and, if the engineer has such small regard for thorough work, the contractor and his foremen and workmen surely cannot be expected to give the construction the greater thought and care essential to satisfactory workmanship.

On the other hand, the extremist who bases his plans on the expectation of perfect materials and workmanship, and allows nothing for imperfections in both, produces designs which, though carried out with all reasonable care, cannot secure satisfactory results. At the same time, he wastes much labor in substantially useless calculation, for considerations of convenience and of uniformity of design do not permit of sectioning exactly in accord with the calculated stresses,

* Kansas City, Mo.

Mr. Harrington.

and knowledge of the imperfections of materials, of the shortcomings of workmen, and of the difficulties of construction, leads to the making of many assumptions and approximations.

Competent and experienced designers, who know and respect the theories involved in the design, but who also give careful consideration to the imperfections of the materials and the practical difficulties of construction, will secure the satisfactory and enduring structures. Mathematical analysis must be supported by sound judgment.

The advocates of the use of high-carbon steel for reinforcing concrete rarely understand its fragility. Any engineer who has had much experience in the shops knows that even medium steel, when dropped from the skids or otherwise roughly handled, is likely to be fractured; and in many instances which have come to the writer's attention, the breakage of high-carbon steel bars in unloading and handling has been so great as to cause serious doubt as to their quality. The bending of reinforcing bars is not a nice job: It is done in haste, and generally by unskilled workmen, often with unsuitable tools; hence breakage, or what is worse, incipient cracks, which escape notice but enlarge under stress in the structure, is sure to result from the use of this fragile material.

In the writer's opinion, the saving in cost resulting from the use of high-carbon steel does not warrant its adoption. The saving is generally estimated to be proportional to the difference between the unit stresses permissible in the various grades of steel, but this is not true, for, as every experienced designer knows, members cannot be proportioned exactly in accord with the stresses in them, and a large quantity of ties, hoops, and spacers are necessary, regardless of the unit stresses used. The saving is not worth the risk, particularly in structures subject to moving loads.

The finish of concrete structures is one of the most serious moot questions relating to them. In choosing a finish, the improvement in appearance is but one of the considerations; the added cost, the greater absorption of water, and the fouling by smoke resulting from the breaking of the cement skin, must be carefully considered. If the structure is situated so that few will see it at close range, or that it will be blackened by smoke, the expense of treating the surface is not warranted. The writer recently examined a viaduct over a railway yard, and it was covered with soot to such an extent that neither the character of the surface nor the material of the structure could be readily determined. In parks, on boulevards, and in other exposed positions, however, where many view the structure daily, bush-hammering, rubbing, floating, or some other satisfactory method of finishing the surface should be used in order to remove ugly form marks and secure uniformity of color and texture. There should be

Mr. Harrington.

no attempt to make concrete resemble stone, but flat surfaces should be broken. Plastering is almost certain to scale off and leave a surface uglier than that left by the forms; and, except possibly by using steel-lined forms, it is practically impossible to secure satisfactory surfaces, except by some mechanical treatment.

No amount of surface dressing, however, will compensate for ugly lines and details, and the use of brick, tile, or plaster ornaments is rarely if ever commendable. The designer of such structures as this must rely for appearances chiefly on his ability to give them form and line suitable to the service to be rendered by the structure and in harmony with its dimensions and surroundings. The selection of the central or distinguishing features and the general treatment of the principal forms and parts of the structure are of vastly more importance than ornamentation or finish of surface.

The writer had too large a part in the design and construction of this viaduct to permit him to extol its appearance. The situation and conditions were unusually difficult, hence careful study was required to secure such esthetic results as the general views of the viaduct disclose.

Mr. Howard.

E. E. HOWARD,* M. AM. SOC. C. E. (by letter).—It is of interest to observe that the mathematical demonstrations by Mr. Mensch and Mr. Slettum alike warrant the conclusion that the approximate methods by which the columns were actually proportioned give results within safe limits. There are refinements beyond which ordinary designing calculations for elastic structures need not be carried, provided there are occasional demonstrations that the approximate methods will vary only within safe limits. The type of arch and abutments suggested by Mr. Mensch has been considered and analyzed for similar cases, although not for this specific one, and the additional material in the arch proper to provide for the greater stresses, the adjustment for foundations, etc., never seemed to be compensated for by the saving in abutments. The foundation difficulty referred to may easily become quite serious.

To answer Mr. Mensch regarding high-carbon steel, the writer can suggest only that the same conservatism which leads most engineers to use medium steel in structures wholly of steel, would prompt the use of the same material in structures partly of steel. The writer has seen high-carbon steel bars, actually by the carload, which had been inspected and accepted at the mill, broken by merely being thrown from the car in unloading. Evidently, the sentence concerning the fastening of bars was not wholly clear, for the bars were tied together principally to hold them in correct position while placing the concrete. Any joining of bars intended to transmit the load from one bar to

* Kansas City, Mo.

Mr. Howard.

another was made by some definite splice, such, for instance, as bolted rope clips.

Mr. Holmes' suggestions for constructing influence lines for extreme fiber stress brings forward another of the frequent cases in which individual preference may enter. In this special instance, the designers felt that to carry out the equations another step and plat the influence in lines of extreme fibers would require more work than the method followed. It is easy to agree with Mr. Holmes that in certain cases the other method would be most desirable.

The writer is most heartily in accord with Mr. Holmes' views concerning the relations between engineers, contractors, and public officials. The absurd and ridiculous laws sometimes controlling public construction have been cause for exasperation to many engineers, although, after all, the rabbit-like timidity of public officials, in fearing to do what they believe to be right, may be equally at fault. Although supposedly representing a whole community, public officials are often prone to listen to the vociferous requirements and the threats or promises of a small group of those who have personal interests to promote, and to forget the great silent majority who are not present. Fortunately, there are occasional very refreshing exceptions.

Mr. Upson's figures concerning the piles are substantially correct, but, as a just concession to the increased percentage of overhead cost due to the reduced quantities, a payment of $2 462 was made to the Contractor. This still left the City about $20 000 ahead of the probable cost of pre-cast piles. The Raymond piles were well suited to the foundation conditions and to the season in which the work had to be done.

The rather unusual requirements of a cement-gun finish, referred to by Mr. Ash, were included as one alternative method of surface finish only after the manufacturers of the cement-gun equipment gave assurances that they could provide such work, and gave an approximate price for it. The experiences of other users of the cement gun, especially on viaducts of the Kansas City Terminal Company, cast serious doubt on the practicability of producing an even, uniform, $\frac{1}{8}$-in. mortar coat over irregular surfaces. It was evident that the extruding corners would not be coated at all, that re-entrant corners and moulds would be filled and rounded, and that the clean lines of the structure would be lost. It is the writer's opinion that the cheap and satisfactory surface finish for ordinary concrete will some day be supplied by some kind of lime-cement wash or paint which can be brushed on, which will have characteristics so similar to the concrete that it will not flake off. Many industrial chemists are studying cements, limes, and mortars, and such a wash should be available at no distant day.

In conclusion, it has been a pleasure to the writer on frequent occasions to hear expressions of pride and satisfaction, from residents

Mr. Howard.

of Kansas City, who provided the money for the viaduct, in the pleasing appearance and evident excellency of construction. The public verdict is that the structure is a success. The old excuse that engineers have been prone to bring forward, that their principals would not provide money for beautiful structures and therefore such structures were ugly, belonged to pioneer conditions, and, as Mr. Green suggests, the United States may fairly be said to have passed that stage. It would be hard to find any one, even of those who paid special assessments for this viaduct, who would ask the return of the small additional percentage which it cost, over the expense of a series of posts and beams which would have carried the load. If the Engineering Profession will lead the way for substantial and beautiful bridges, public sentiment will follow fast enough.

The writer desires to express his cordial appreciation for the courteous words of approval of his paper by those who discussed it; and, on behalf of all the engineers connected with the design and construction of the Twelfth Street Trafficway Viaduct, for their compliments in regard to the structure.

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Paper No. 1358

THE AUTOMATIC VOLUMETER*

By E. G. Hopson, M. Am. Soc. C. E.

With Discussion by Messrs. George W. Booth and E. G. Hopson.

Synopsis.

This paper describes an apparatus intended to gauge the flow of fluids by the collection of a proportionate part of the flow, or its equivalent, in a small vessel where it can be readily measured at any time. This result is accomplished by the use of very small orifices for the purpose of regulating the discharge into or out of the collecting vessel, and other special arrangements, whereby the pressure head under which the discharge into the collecting vessel takes place is at all times the equivalent, or a constant ratio, of the velocity head of the liquid or gas being measured.

Practically all measurements of flow of fluids on a large scale are difficult and expensive. As soon as one gets beyond the small service meter for water or gas, very expensive apparatus is generally involved. The accuracy of any apparatus with moving parts or mechanism is, moreover, frequently open to question, due to the difficulty of constructing any such mechanical device with a constant coefficient of

* Presented at the meeting of December 1st, 1915.

flow. To express the latter point in other words, the usual mechanical meter does not record with equal precision at different rates of flow; thus an instrument that will record correctly at a given rate will usually record too small for lesser rates, and *vice versa*.

The importance of a cheap and reliable apparatus to record the flow of water in small irrigation laterals, for individual services, induced the writer to take up the study of developing this device. The result indicates that it may be of equal utility to city water-works systems or to industrial plants, particularly where it is desired to ascertain the duty of pumping machinery, or to determine the use or waste of water in a pipe-distributing system. Its application to all forms of flow measurement necessarily follows as a matter of course; thus, if it can be used for water, it is equally available for oil, solutions of various kinds, or even for gas or steam. All fluid flow is subject to the same law that governs velocity and its relation to head, so that the application, to any fluid, of a device dependent on this factor only is a matter of detail.

In this discussion no attempt is made to describe the application of the device for other purposes than the measurement of the flow of water, and the experiments and tests herein described only have reference to water flow and measurement thereof.

The law now requires that on Government irrigation projects payment for maintenance shall be based on measured quantities of water delivered to the individual. Under present conditions, it is practically impossible to determine quantities; only a rough guess, in which one is guided to some extent by the ditch rider's observation and notes, is possible.

For irrigation use, probably one of the most dependable devices for measuring individual service is the Dethridge meter, which is merely a paddle-wheel operating a recording counter and has the advantage of actually measuring the displacement by each revolution of the wheel. Other methods of measurement, by weirs and modules, are inaccurate, save with apparatus and attention entirely prohibitive for ordinary purposes. The Venturi meter, as recently adapted for irrigation use, appears to be the best instrument on the market, so far as scientific accuracy and freedom from accidental interruptions are concerned; but it involves elaborate recording mechanism, with detailed reduction of results, and is expensive.

The volumeter herein described operates by the velocity head of the fluid which is being measured. The velocity head is communicated by a pipe leading from the fluid being measured to the volumeter. Not only is velocity head communicated, but a very small proportion of the actual flow is diverted by this pipe into the apparatus. This proportion is so small, however, and the velocity of flow in the pipe is so insignificant, that there are no appreciable losses of head by friction in the pipe to disturb the basic plan of operation by velocity head. A return pipe from the volumeter to the stream carries away the surplus liquid created by the influent flow.

So far as the volumeter is concerned, it operates by head or pressure. It will operate under the influence of any head or pressure, no matter how derived. Although it has been termed a volumeter—inasmuch as it measures volume—it actually is a velocity meter or pressure meter, in the sense that velocity, or pressure due to velocity, is its operating force. Therefore, it is absolutely essential to its successful operation that it shall be erected in such a way as to be subject to operation by the head due to velocity alone.

The apparatus may be attached to a pipe or conduit, with communication through a Pitot tube, or at submerged orifices, or on a short pipe or box flume of varying cross-section, or at an angle or bend in a pipe or conduit, all as illustrated on Fig. 9. Other methods of attachment will doubtless occur to the reader. The methods, however, are unimportant, so far as the operation of the device is affected, so long as they are consistent and such that the head communicated to the volumeter is a constant factor of the velocity head. The latter is the essential condition.

The most difficult feature of the apparatus to work out was an arrangement whereby the head, under which the interior jet operates, is kept identical with, or is a constant factor of, the velocity head of the stream. It is necessary for the apparatus to be absolutely automatic, and its response to the most minute changes of head to be immediate and of the most delicate sensitiveness. This requirement has been met. The influent entering the upper part of the lower vessel (Fig. 1) creates a pressure at its point of entry equal to the sum of the static and velocity heads of the entering water, the frictional losses in the influent pipe and orifice being negligible, due to their great size and the very low velocities in them. The lighter

medium (liquid or gas) contained in the lower vessel is thus put under pressure. The upper vessel is connected with the flowing water in such manner that only the static head is communicated to it. The flow through the small orifice connecting the two vessels is, therefore, for all practical purposes, that due to the velocity head, or a constant factor of the velocity head, only.

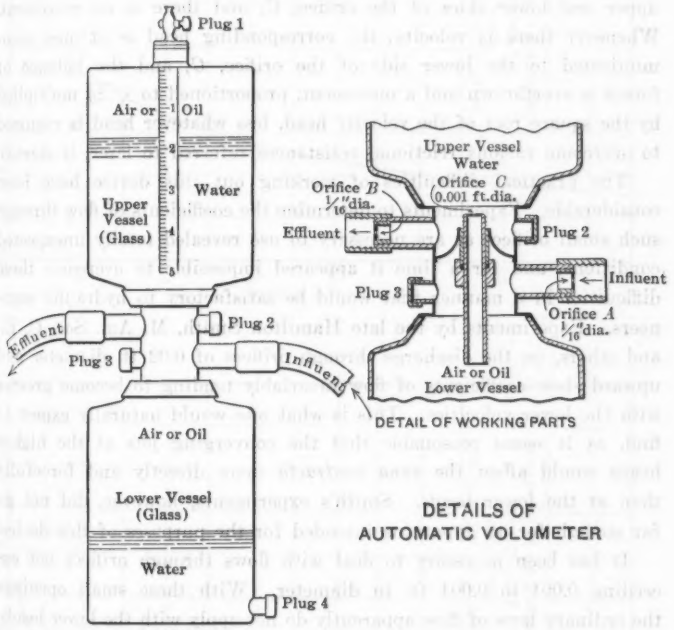


FIG. 1.

The operating details will be fully understood by reference to the detailed drawing, Fig. 1. The influent enters the lower vessel or container at its upper part through an orifice at *A*. The pressure communicated is due to the static head or pressure of the water being measured plus the velocity head due to the rate of flow. This pressure is communicated to the medium (air, oil, or whatever may be used) in the lower vessel, and in turn is transmitted through the

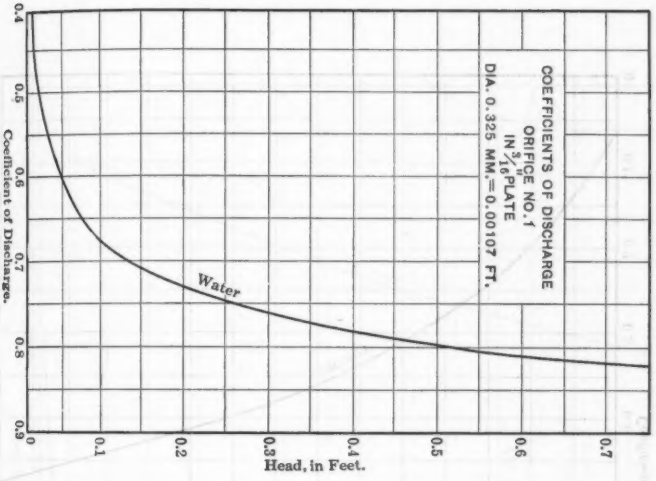
medium to the lower side of the point of connection with the upper vessel at the orifice, *C*.

The water in the effluent pipe is only under the pressure due to the static head of the stream being measured. This pressure is communicated directly through the water in the lower part of the upper vessel down to the upper side of the orifice, *C*. When there is no velocity, the static head of the stream is communicated to both the upper and lower sides of the orifice, *C*, and there is no movement. Whenever there is velocity, the corresponding head is at once communicated to the lower side of the orifice, *C*, and the balance of forces is overthrown and a movement, proportioned to $\sqrt{2g}$ multiplied by the square root of the velocity head, less whatever head is required to overcome various frictional resistances referred to later, is started.

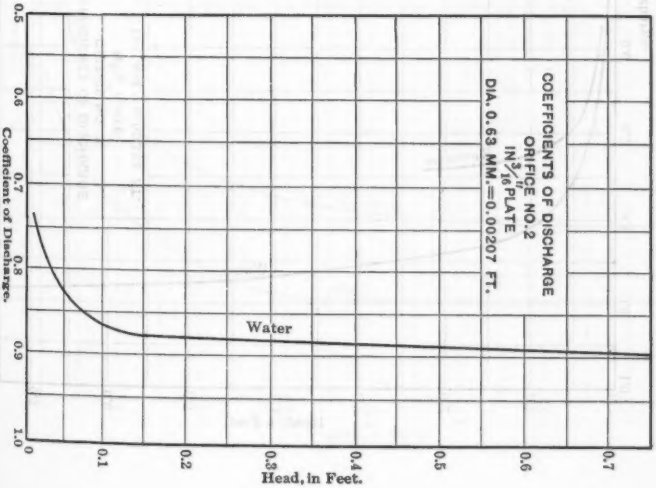
The practical difficulties of working out this device have been considerable. Experiments to determine the coefficients of flow through such small orifices as are necessary to use revealed totally unexpected conditions, and for a time it appeared impossible to overcome these difficulties in a manner that would be satisfactory to hydraulic engineers. Experiments by the late Hamilton Smith, M. Am. Soc. C. E., and others, on the discharge through orifices of 0.02 ft. diameter and upward show coefficients of flow invariably tending to become greater with the lower velocities. This is what one would naturally expect to find, as it seems reasonable that the converging jets at the higher heads would affect the *vena contracta* more directly and forcefully than at the lower heads. Smith's experiments, however, did not go far enough for an apparatus intended for the purposes of this device.

It has been necessary to deal with flows through orifices not exceeding 0.001 to 0.004 ft. in diameter. With these small openings the ordinary laws of flow apparently do not apply with the lower heads, as can be seen by reference to the curves of coefficients, Figs. 2 to 6.

Another marked difference between results obtained with these small orifices and with the larger ones is that apparently the former have distinctly greater coefficients under certain heads than the latter. The largest coefficient noted in Smith's experiments with small orifices was about 66% of the theoretical velocity, and 90% appears to be the rule for the minute orifices referred to herein when operating under heads larger than about 0.2 ft. This is an interesting scientific fact, hard to explain, but it does not seem to be open to question.



Coefficient of Discharge.
FIG. 2.



Coefficient of Discharge.
FIG. 3.

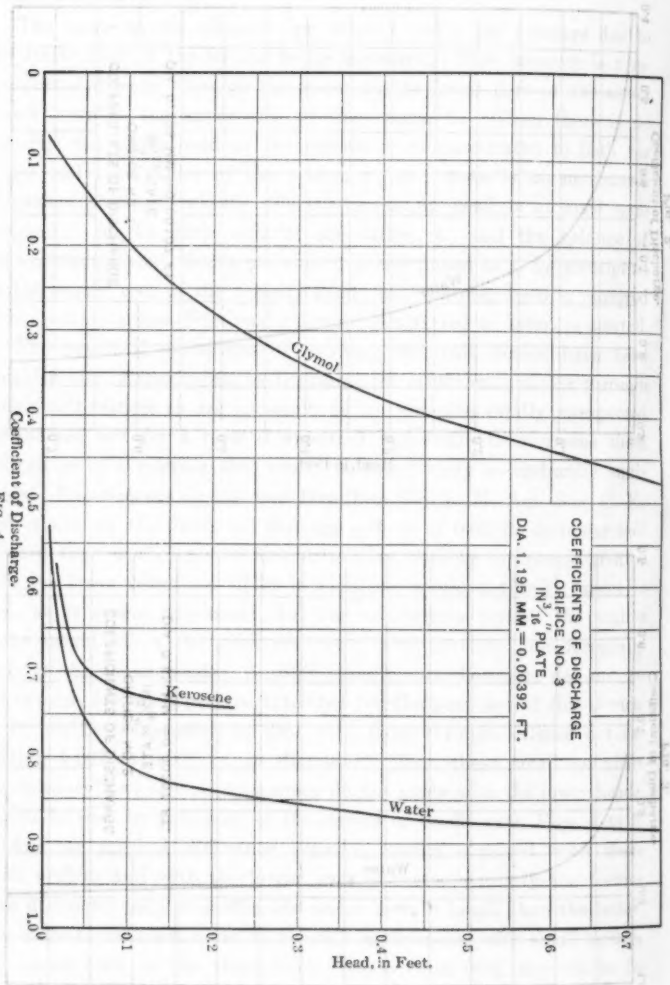

COEFFICIENTS OF DISCHARGE
ORIFICE NO. 3
IN 3/16" PLATE
DIA. 1.195 MM.=0.00392 FT.
Glymol
Kerosene
Water
Coefficient of Discharge.
Head, in Feet.
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7

Fig. 4.

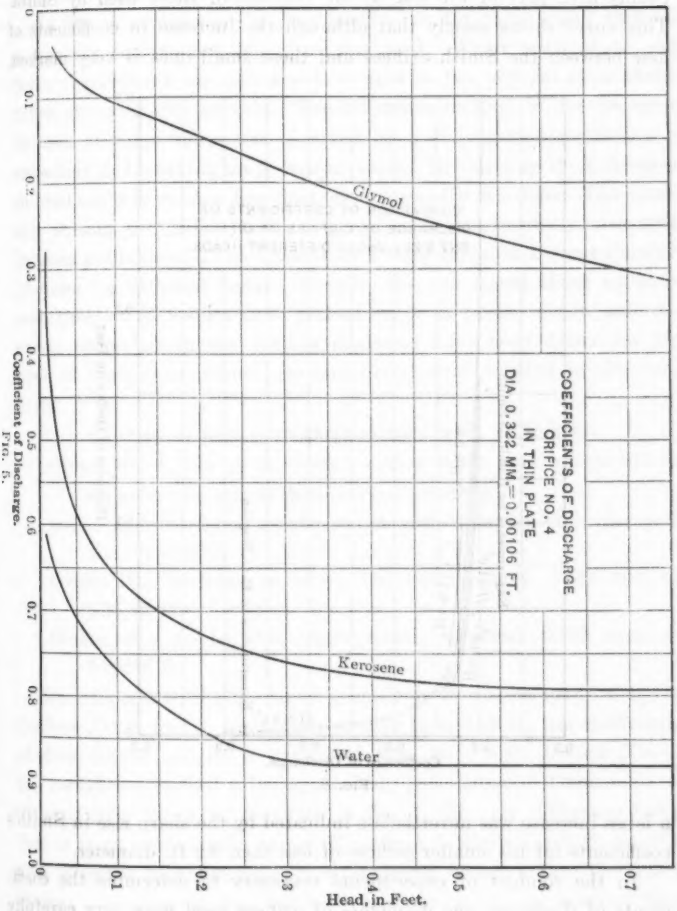

COEFFICIENTS OF DISCHARGE
ORIFICE NO. 4
IN THIN PLATE
DIA. 0.322 MM.=0.00105 FT.
Glymol
Kerosene
Water
Coefficient of Discharge.
Head, in Feet.
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0

FIG. 5.

The curve of coefficients, Fig. 6, shows the coefficients of discharge of some of the orifices experimented on by Smith as compared with those referred to herein, and of which the largest is only about one twenty-fifth part of the size of the smallest of those used by Smith. This curve shows clearly that although the increase in coefficients of flow between the Smith orifices and these small ones is very marked,

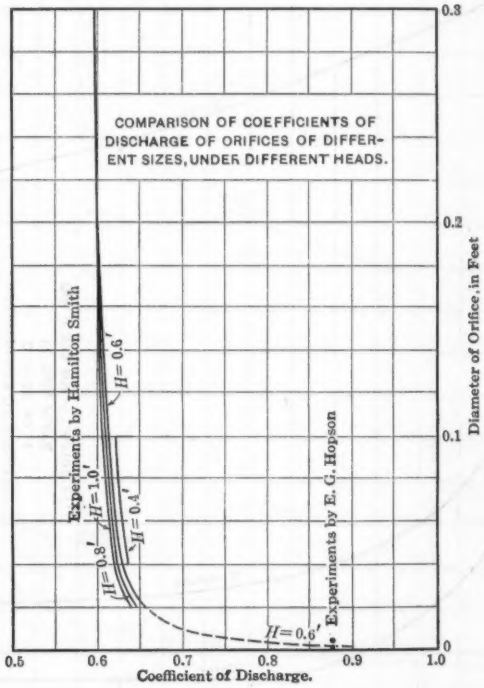


FIG. 6.

a large increase was nevertheless indicated by the sharp rise in Smith's coefficients for his smaller orifices of less than 0.2 ft. diameter.

In the conduct of experiments necessary to determine the coefficients of discharge, the diameters of orifices used were very carefully calibrated by using tapering needle points of which the diameters at different points in their length had been determined by micrometer measurements. Orifices were at first made in a brass plate, $\frac{3}{16}$-in. thick,

and, for the purpose of reducing contraction to a minimum, the up-stream approach was turned—in the thickness of the metal—to an easy curve, so that the orifice itself would be in the position of the *vena contracta*, as nearly as could be determined.

In the determination of flow coefficients for these small orifices, a specially constructed apparatus was used. Water from above and below the orifice being tested was brought to two vertical glass gauge tubes, arranged side by side. The difference in level of the meniscus in each of these tubes was observed by a sliding gauge carrying a cross-hair and working on a vertical scale. In this way the difference in head could be read to less than one-thousandth of a foot. The quantity passing the orifice under a given head was carefully measured in glass graduates. These experiments were repeated a great number of times for different heads. Usually, the run lasted about an hour each time. The results were platted on cross-section paper, and the curves appearing in the various diagrams have been drawn on the basis of these tests, which, for simplicity, were omitted in the final draft.

Four different orifices were experimented with, as follows:

Orifice No. 1.—In $\frac{3}{16}$-in. plate; orifice 0.325 mm. or 0.00107 ft. diameter; up stream with curved approach.

Orifice No. 2.—Same as above, but with diameter of orifice 0.63 mm. or 0.00207 ft.

Orifice No. 3.—Same as above, but with diameter 1.195 mm. or 0.00392 ft.

Orifice No. 4.—In thin sheet metal; diameter 0.323 mm. or 0.00106 ft.

By reference to the curves of coefficients it will be found that for Orifices Nos. 2 and 3, with heads greater than 0.15 ft., the coefficients of flow ranged usually a little less than 90. With Orifice No. 1, the coefficients seemed to be appreciably less, but with Orifice No. 4, of practically the same diameter as No. 1, the only difference being that it is in thin plate, a range of coefficients almost as high as with Nos. 2 and 3 was obtained. It is apparent, however, that there is a sharper reduction in the low-head coefficients for the smaller than for the larger orifices.

With heads of 0.015 ft. and greater a fair degree of accuracy was possible with the experiments; for heads of less than this it was ex-

tremely difficult to obtain absolute accuracy. By careful repetition of experiments and extension of the curves obtained under the larger heads, it is believed, however, that the results as shown on the curves, even for the lowest heads, are very close to the truth.

The relatively small coefficients of flow under the smaller heads, which appear to be absolutely established by these experiments, necessarily introduced a disturbing element into the measuring device. It was apparent that the accuracy of the measuring device must depend on coefficients of flow for these small orifices being of reasonable constancy. The experiments, however, clearly show that for the smallest orifice the coefficients are reduced from 88% for a head of 0.3 ft. to about 60% for a head of 0.01 ft., a reduction of about 33%, an amount that would render the apparatus useless for scientific purposes.

The difficulties created by the varying coefficients of the small orifices have been overcome in the volumeter by a special feature which increases slightly the operating head, as will be referred to later.

It is an interesting fact that no advantage in flow coefficient was apparent from the curved approaches of Orifices Nos. 1, 2, and 3, in $\frac{3}{16}$-in. plate. The same, or even greater, discharges were found when the plate was reversed and the curved approach was turned down stream; moreover, Orifice No. 4, in thin plate without any curved approach, gave distinctly higher results than Orifice No. 1.

In addition to experiments with water, runs were made using oils. A careful investigation of oils on the market was made to find one having a viscosity which did not change very materially under changes of temperature such as would be encountered in a water-measuring device under working conditions (see Fig. 7 for viscosity tests). The oils most nearly approaching these conditions were kerosene, a petroleum product known commercially as white neutral oil, and another petroleum product known as glymol. The latter is a thick, clear oil, used generally for medicinal or surgical purposes. It has a viscosity ranging from about two to five times that of water or kerosene, within a range of temperatures between 32° and 100° Fahr. A series of careful runs was made with glymol and kerosene, the results being shown on the accompanying diagrams. It will be noted that the coefficients of flow for glymol ranged very low, the highest being less than 30% for a head of 0.6 ft. It is clear that no satisfactory results can be obtained by using the flow of this medium

in a measuring device. Kerosene gave results very similar to those of water, but about 10% less in amount.

A difficulty of appreciable moment in connection with this device is that involved in inducing a current or jet of liquid or gas of less specific gravity than water to discharge into the latter without sacrificing any of the head due to velocity of the stream which is being measured. It is to be borne in mind that the apparatus must necessarily operate under extremely small heads, such, for instance, as 0.01 or 0.02 ft., as due to velocities as low as 1 ft. per sec. or less. Therefore, there is no energy to spare in the velocity head of the stream being measured, all of this head being required for the purpose of recording; in other words, the operation of the apparatus itself must be carried on with power entirely outside of the static or velocity head of the stream.

If a vessel containing water is superimposed on a tight vessel containing air, and if these be connected by an orifice less in diameter than about 0.011 ft., it will be found that the water will not flow down through the orifice after the air in the lower vessel has been compressed to a density sufficient to resist the pressure of the superimposed water. Surface or skin tension of the water itself, no doubt, has much to do with this resistance, but, whatever the reasons are, it will be found that the water pressure is balanced by the air pressure plus this frictional resistance. If the orifice is made larger in diameter than 0.011 ft., there will begin to be a tendency for the air and water to pass each other through the orifice, the air escaping by bubbles and marked disturbance of the water. With smaller orifices, however, there is no passage of air and water, and a condition of equilibrium is maintained.

If oil is substituted for air in the lower vessel similar results will be observed, but to a much more marked degree, depending on the viscosity of the oil. The repulsion between oil and water and the surface tension of each medium creates a balance of forces, even with connecting orifices of very considerable size between the vessels, so that the heavier medium will not sink into and displace the lighter one. This refusal of the different media to pass each other through small orifices is a feature of importance in working out the measuring device herein referred to.

It was found by experiment, moreover, that a very appreciable force was necessary to break the seal of surface tension and induce a flow between differing media through the small orifices required in this apparatus. The amount of this force differs with the size of the orifice and the character of the medium. With an orifice 0.001 ft. in diameter, a head of nearly 3 in. was necessary to overcome this resistance and induce air to pass upward into water. With oils the resistance is less than with air, but in all cases there is an appreciable resistance to be overcome.

From the foregoing it will be understood that a substantial operating head or force must be provided from other sources than the static and velocity heads of the flowing stream. In the volumeter, this force has been obtained from gravity operating between the two vessels, in which the heavier medium is at the top and consequently tends to sink to the bottom and displace the lighter medium there contained. The apparatus provides means whereby the force of gravity is exactly controlled so as to furnish operating power and to apply the necessary correction to the discharge coefficient of the controlling orifice.

Referring again to the reduced coefficient of the flow with these small orifices under low heads, a study of the curves of coefficients seems to indicate clearly that the reduction is largely due to some obscure retarding force, such as surface tension, which actually absorbs a portion of the operating or velocity head applied, and is, to a considerable extent, if not entirely, a constant in itself. Thus, with heads as low as 0.01 ft., the coefficient of flow may be as low as 50%, which would indicate the velocity of the jet to be about 0.4 ft. per sec. If the coefficient had been about 90%, as in the case of the higher heads, the velocity would have been about 0.7 ft. per sec. Thus the loss in velocity due to surface tension or some similar cause is about 0.3 ft. per sec., an amount which would correspond with an approximate loss of head of about 0.007 ft. Thus the addition of a constant correction of some small amount like this had been found practically to eliminate the variant in the discharge coefficient for the small orifices, and to give a resulting accuracy to the device, which is forcibly brought out by the curve of discharge shown in comparison with the curve of theoretic velocities and discharge in Fig. 8. These results are of the highest practical importance.

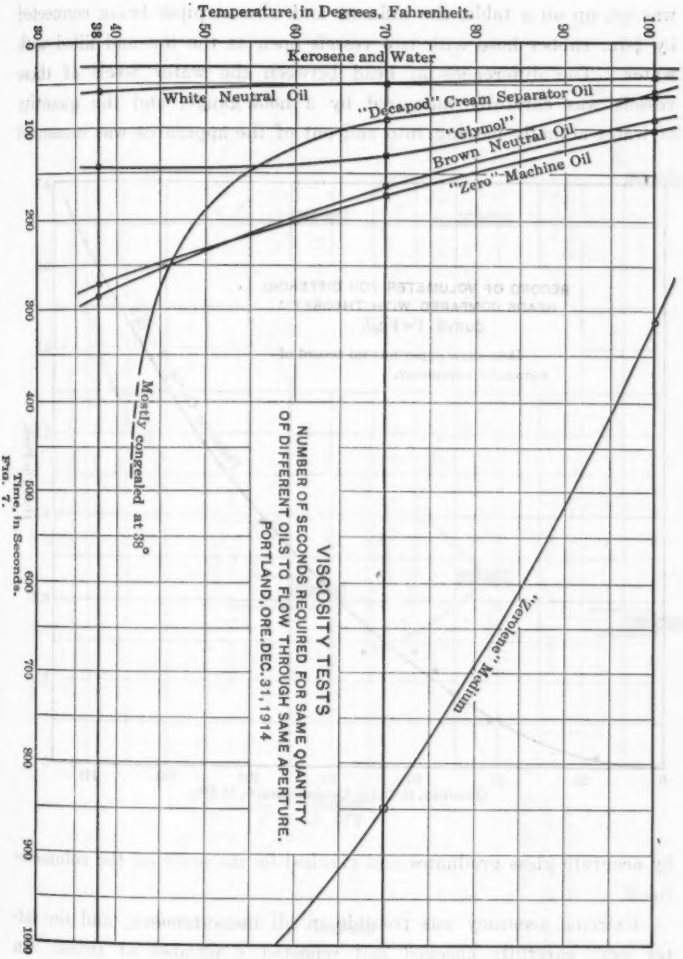

Temperature, in Degrees, Fahrenheit.
32
40
50
60
70
80
90
100
0
100
200
300
400
500
600
700
800
900
1000
Time, in Seconds.
Kerosene and Water
White Neutral Oil
"Decapod" Cream Separator Oil
"Glymol"
Brown Neutral Oil
"Zero" Machine Oil
"Zerolene," Medium
Mostly congealed at 38°
VISCOSITY TESTS
NUMBER OF SECONDS REQUIRED FOR SAME QUANTITY
OF DIFFERENT OILS TO FLOW THROUGH SAME APERTURE.
PORTLAND, ORE. DEC. 31, 1914

FIG. 7.

In the determination of discharge of the volumeter for different heads, as given in Fig. 8, the following method was used: The apparatus was set up on a table, the influent and effluent pipes being connected by $\frac{3}{4}$-in. rubber hose with two vessels open at the top and filled with water. The differences in head between the water levels of these vessels was carefully measured by a hook-gauge, and the quantity of water actually passing into and out of the apparatus was measured

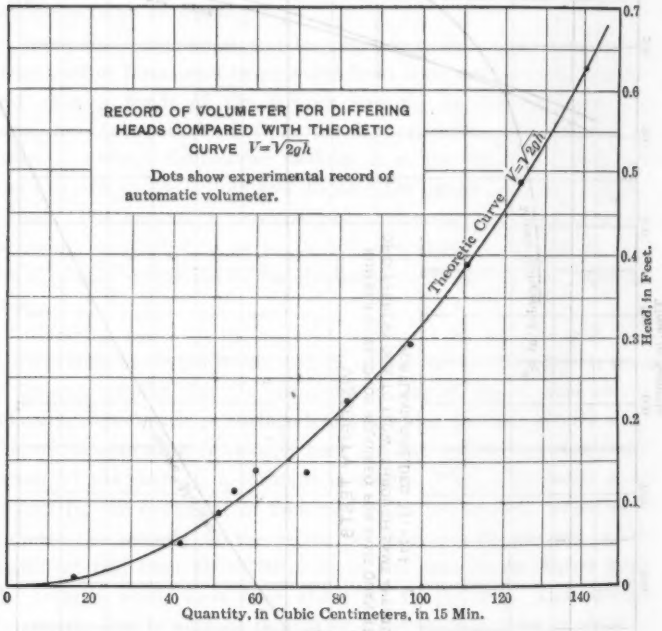
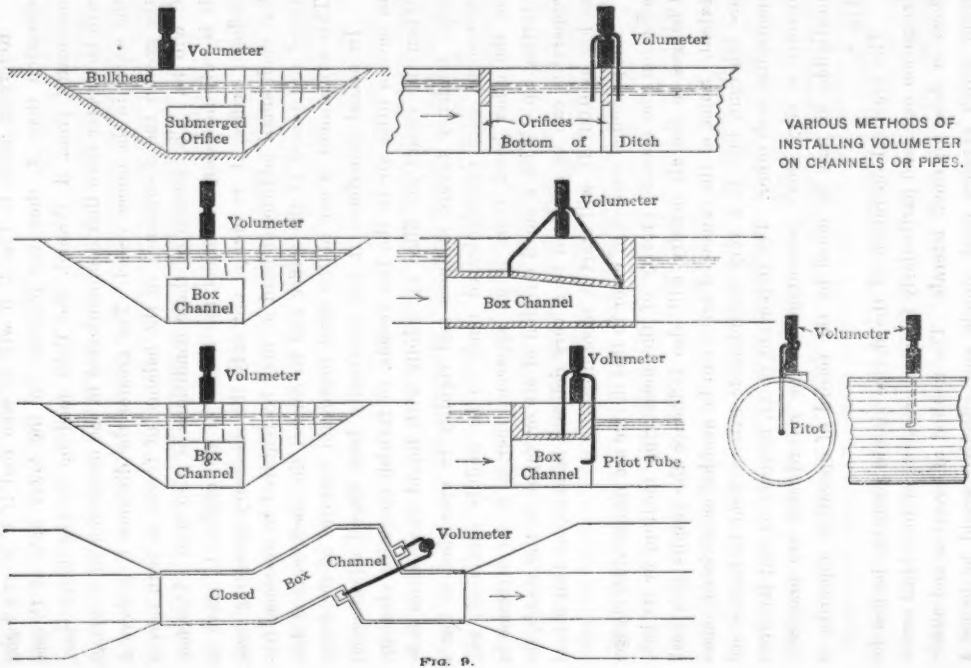


FIG. 8.

by accurate glass graduates and checked by the scale on the volumeter itself.

Extreme accuracy was possible in all measurements, and the latter were carefully checked and repeated a number of times. On Fig. 8 the quantities passing are expressed in cubic centimeters per 15 min. This is a purely arbitrary measurement, and might as well be expressed in some other form; actually, the quantities measured



VARIOUS METHODS OF INSTALLING VOLUMETER ON CHANNELS OR PIPES.

FIG. 9.

were the result of much longer runs than 15 min., usually being 1 or 2 hours in each case.

Referring again to Fig. 1, it will be seen that if *C* is at a higher elevation than *A*, then the pressure on the lower side of *C*, communicated from *A* through the light medium in the lower vessel, will be greater than that communicated to the upper side of *C* through the water in the upper vessel. The greater the distance *C* is above *A*, the greater the discrepancy in the balance of forces on both sides of *C* will be. The reverse of these conditions is also true. Therefore, it is only necessary to adjust the height of the orifice, *C*, above the influent orifice, *A*, to obtain, not only the necessary operating force, but such small additional head as may be required to overcome frictional resistances at the orifice and to correct the coefficients of flow. This adjustment is one of the most important features of the device.

In actual experience it has apparently been found advantageous to have a little air in the two vessels, to furnish operating elasticity. When the vessels are filled up solidly with liquid there seems to be some tendency toward stiffness and rigidity in operation, so that a movement once communicated tends to be unduly continued; and, on the other hand, there is an apparent inertia to be overcome in starting up. With a small cushion of air or vapor in the vessels, the apparatus can be adjusted to a very delicate balance, so that a head as low as 0.001 ft. will start it recording.

The principal sources of error in this device are temperature changes affecting the specific gravity of the measuring medium or its bulk. Changes in the air density will also influence the results to a small extent. Some of the causes of error can be avoided or obviated; others are inherent, but, to a very considerable extent, tend to balance and neutralize each other. The important point, however, is that the errors involve a very small percentage of the record and are practically negligible, as may be noted by the record of operation appearing on Fig. 8.

The practical operation of the device should probably be first for irrigation uses, and particularly for measurements of individual service flows or flows in small laterals. The method of erection and attachment which appears most simple and effective would be to use a section of box flume or pipe, a few feet in length, at the point of measurement, or two submerged orifices, as shown on Fig. 9. The

device would be connected with the stream by permanent iron pipe connections and protected by a small box cover. Recharging of the apparatus could be accomplished without disturbance other than merely emptying one vessel into the other.

The device records volume at any given time. No record is made of the various changes in rates of flow during the period of observation. If this is desired, however, it is a mere matter of adding clock-work recording mechanism. A continuous roll of sensitized paper passed by the gauge will enable a photographic record of variations in flow to be obtained. Such a record will be a mass-curve of the flow during the period of observation.

The device, as applied to irrigation use, has the advantage that it is independent of such matters as drifting sand, weeds, or any floating or suspended trash. Nothing in suspension will pass into the influent pipe, which has practically no velocity of flow and has, moreover, an upward course. Solid matter if placed in the influent pipe would in fact settle back into the stream. Even if suspended or floating material should get into the measuring vessels, it would not pass through the controlling orifice on account of the internal arrangements. This has been put to very severe test in the experimental run, as the water used in this was full of impurities and suspended matter.

An apparatus like this, of course, must be set with skill by one who understands the principles of its operation. With proper setting, its further operation is so simple and automatic that any ditch rider or mechanic of average intelligence will be able to reset and read it each day or whenever necessary. The device is as near a self-contained and self-operating one as can well be imagined.

The device should be cheap. It ought to be made and sold for a few dollars, which should cover everything, including all the necessary pipe and fittings. This should place it within the reach of all.

For use on water-works systems or for industrial purposes its simplicity and cheapness should render it valuable and enable a more intelligent understanding to be had of many operating conditions that have hitherto not been fully understood owing to the cost of large meters and the skill required in their operation and the deduction of results.

DISCUSSION

Mr. Booth. GEORGE W. BOOTH,* M. AM. SOC. C. E. (by letter).—This very ingenious and simple device should have a wide field of application, provided it proves in service to be as generally applicable to the various conditions of fluid flow as Mr. Hopson's tests lead him to believe. As he observes, one of the obstacles at present met in providing measuring devices of reasonable accuracy is the cost of putting in and maintaining the apparatus.

There appears to be no reason for not using this device in measuring the flow in pipe distributing systems, provided the velocity of flow is sufficient to start the recording apparatus, as it usually will be in such systems; Mr. Hopson states that the velocity head required for this is as low as 0.001 ft.

The writer has been interested to determine whether the volumeter could be used on fire line services to buildings equipped with automatic sprinklers or stand-pipes. If it could be certain of use solely for fire protection, there would be no call for a meter, because this use occurs only at very infrequent intervals, and the total consumption is comparatively small; however, the experience of some water departments in connection with the illicit use of water for other purposes has been such that the placing of some form of meter is being required, and the cost of a satisfactory one is a considerable objection, especially in the smaller buildings.

The most common size of fire line service is 6-in., with a 4-in. pipe for buildings having less than about 6 000 sq. ft. of floor area, and an occasional service of larger size. If the minimum velocity head at which the apparatus will register is 0.001 ft., corresponding to a velocity of about 0.3 ft. per sec., flows of less than about 22 gal. per min. in a 6-in. pipe, and about 10 gal. per min. in a 4-in. pipe, will pass unrecorded. A meter not capable of measuring these smaller flows would hardly be satisfactory for this service, and it would probably be necessary, as is the present practice with other types of meters, to provide for measuring them through a by-pass, with an arrangement to divert the flow to the main channel automatically at such times as a large supply and small loss of head are required.

Mr. Hopson. E. G. HOPSON,† M. AM. SOC. C. E. (by letter).—Mr. Booth suggests the possible usefulness of the automatic volumeter for metering individual fire mains supplying sprinkler systems, and it would seem that this proposition may have considerable merit.

With regard to these fire mains, the conditions, as understood by the writer, are as follows: Such mains are intended for use solely for

* New York City.

† Portland, Ore.

Mr. Hopson.

fire protection, and, ordinarily, are merely filled with water under pressure which is not drawn upon, save possibly for occasional flushing or test. The quantity of water used strictly for fire purposes is negligible, under ordinary conditions, where no fires occur, and, even when they do, it is relatively small as compared with the daily use of water for domestic or mercantile purposes. There is no object in measuring the water used strictly for fire purposes, because its quantity is insignificant at all times, and the idea of economy or limitation in its use is not to be considered.

The illicit use of water from these fire mains has always been a standing annoyance, and has led to the requirement referred to by Mr. Booth that these services be metered. That, again, introduces another difficulty, as a meter for a 4-in. or 6-in. main is not only expensive, but is not suited for the detection and registration of illicit drafts, which, though small in each case, are considerable in the aggregate, owing to their frequency. In practice, this difficulty has been met, in part by the introduction of a small metered by-pass which permits the complete registration of all such uses, but has the disadvantage that the main supply is necessarily cut off, and dependence must be placed on some automatic process of throwing in the main during an emergency. From the fire protection standpoint, a system dependent on any automatic control has decidedly objectionable features.

As Mr. Booth correctly points out, if the volumeter can only operate under a velocity head of 0.001 ft. and upward, considerable quantities of unregistered water might pass through a 4 or 6-in. main, if dependence was placed on the apparatus. This estimate is that as much as 22 gal. per min. on a 6-in. pipe, or 10 gal. per min. on a 4-in. pipe might go undetected.

The exact head necessary to start the volumeter recording is not known, no special test having been made for that purpose. The writer has found, however, that it will start and stop under changes of head as small as 0.001 ft. This is a very small head, and perhaps it will be well to assume for present purposes that it is the limit of accuracy. Such a head corresponds to a velocity of about 3 in. per sec. If the fire main can be made to discharge through a reduced cross-section so that this velocity will correspond to a relatively small discharge, then measurement by the volumeter, taken at the contracted section, may be possible and may be sufficient for the purpose in view.

It is suggested, therefore, that a reducer be inserted in the main, possibly in the manner indicated in Fig. 10. This shows a 4-in. main reduced to a 1½-in. throat in about 1 ft. of distance and then correspondingly expanded to the original diameter. The reduced area of cross-section is about one-seventh of the full section, and the velocity at the throat is seven times that in the main. Assuming a working

Mr. Hopson.

velocity of 6 ft. in the fire main, or a discharge of 230 gal. per min., there would be a loss of head (due to the reducer) of only about 3½ ft. or about 1 lb. pressure. Thus, for all practical purposes of fire protection, the fire main would be unthrottled and wholly free from objectionable automatic control.

The volumeter could be connected with the reducer by the influent pipe, *a*, and the effluent, *b*, similar in manner to the connection of the registering apparatus of a Venturi meter. It might equally well be connected by a Pitot tube at the throat as at *a'* and a piezometric connection at *b'* below the reducer, for the effluent. Such an apparatus, in a 4-in. main, would detect and register uses of water as small as 1.4 gal. per min.; or in a 6-in. main, a little more than double that quantity.

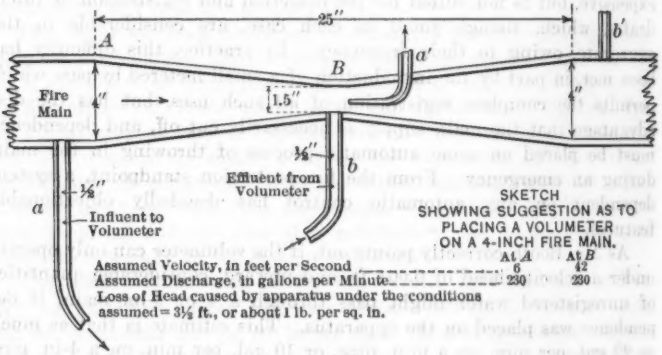


FIG. 10.

Though this minimum might still exclude very small drafts, or even small constant leaks, it would show clearly all the ordinary drafts likely to be made in a mercantile establishment, such as the drawing of a pail of water, the use of a hose, or similar details.

The limitation of the apparatus with reference to minimum measurement is governed by the maximum practical velocity at the throat. This has been assumed at 40 ft., or perhaps a little more, which apparently is possible without any sensible sacrifice of working pressure in the fire main. If, on the other hand, there is an ample sufficiency of pressure in the fire main, so that as much as 20 lb. pressure can be sacrificed at the reducer, it might then be possible to contract the diameter of the throat still more, and thus correspondingly increase velocities and secure a measurement of flow as low as about ¾ gal. per min. on a 4-in. main.

An advantage of such an arrangement is that it is good from the fire-protection standpoint, and the cost should be very small. The

25-in. reducer would be cheap, and the volumeter and connections should cost only a few dollars. The apparatus has no working parts, and will not get out of order by disuse, but it would be necessary to protect it from frost. Mr. Hopson.

One objection to such an arrangement, called to the writer's attention by Mr. Booth, is the fact that the contracted section of the main has already been objected to by underwriters in the case of the Venturi meter, because of the fear that foreign matter, such as a block, or a stone, or other article, unwittingly left in the pipe when it was laid, might become lodged in the throat and entirely block the system. This objection, the writer believes, has met with considerable support, but for reasons that do not appear to be well founded. If there is any real danger to be feared on this score, it can be absolutely removed by the introduction of a coarse screen in the main above the contraction. Such a screen, having a mesh of heavy wire, or rods, say, at 1-in. intervals, would prevent any article large enough to block the throat from reaching it, and would hold it at a point where its blocking effect on the main would be inappreciable and where it could be removed.

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Paper No. 1359

THE ASTORIA TUNNEL UNDER THE EAST RIVER FOR GAS DISTRIBUTION IN NEW YORK CITY*

By John Vipond Davies, M. Am. Soc. C. E.

With Discussion by Messrs. F. Lavis, Milton H. Freeman, W. H. Bradley, James F. Sanborn, Edward Wegmann, R. C. Kellogg, W. W. Brush, Thomas H. Wiggin, William Cullen Morris, and John Vipond Davies.

Synopsis.

The object of this paper is to describe in general the design and construction of a tunnel of large magnitude, but generally simple in character, and to treat more particularly certain difficulties due to physical conditions encountered and methods used in overcoming them.

The work described consisted of sinking two shafts, 242 and 277 ft. deep, respectively, and driving a tunnel, 18 ft. on the vertical and 19 ft. on the horizontal axis, between these shafts, a distance of 4 662 ft., through rock under a river with a depth of water of 100 ft. The main problem was to drive through a zone of decomposed rock extending on the line of the tunnel for a distance of 450 ft.

The paper is divided into fifteen parts, as follows:

1.—History of project and economic reasons for removal of gas plants from Borough of Manhattan to remote districts, by providing transmission of gas through a tunnel under the East River.

2.—General description of the construction.

3.—Description of the plant.

* Presented at the meeting of October 6th, 1915.

4.—Sinking of the shafts, 30 and 42 ft. external diameter, respectively.

5.—Tunnel driving through various rocks under normal conditions. Geology of line of tunnel. Details of work accomplished, and rates of progress.

6.—Tunneling through decomposed rocks under heavy water pressure, together with a geological description of the sheared contacts and conditions as actually found. In this part is described the method of thorough exploration of the rock formation by test drilling in advance and the consolidation of decomposed material by cement grouting to produce a rock formation through which the tunnel could be safely advanced by ordinary blasting. It also describes the methods of drilling, both with percussion and diamond drills, in preparation for grouting; the practical application and extent of the grouting operations; the use of buttresses to face rotten rock to permit of grouting; and the use of emergency bulkheads to secure the safety of the work and the employees.

7.—Flooding of entire work by breaking out of seam before it was securely grouted and controlled.

8.—Unwatering the tunnel and finally plugging and securing the source of water inlet and solidifying the rock; the methods considered and those adopted; the use of the air lift; the condition of the Portland cement after grouting; and the final completion of the tunnel excavation.

9.—Cast-steel lining used through the bad rock section. Assumptions as to the design of the lining, the details, and the erection.

10.—Drainage of the tunnel.

11.—Ventilation of the tunnel.

12.—Lighting of the tunnel.

13.—Triangulation and alignment; methods used and results obtained.

14.—Quantities of items making up the work; and details of costs of all work executed under normal conditions.

15.—The laying of the gas mains in the tunnel. Two lines of 72-in. cast-iron pipe, each pipe length weighing 26 000 lb. Heaviest cast-iron pipe ever manufactured. Special machinery designed and used for laying pipe. Special machine designed and made for testing joints of pipes as laid. Jointing and caulking of pipe mains.

The writer has endeavored to give a comprehensive description of the work as a whole, and to emphasize the portion of the paper covering the operation of passing through the zone of bad rock. It is hoped that the photographs and drawings which accompany the paper will assist in making clear the actual conditions encountered in the field.

1.—Historical.

About 35 years ago, before the general introduction of electricity, the production of gas for illuminating and heating purposes within the City of New York had reached such a point, with reference to the occupation of lands for manufacturing plants and local interference with city development, as to have made it desirable to consider the removal of these industries from the densely populated sections of the city.

Consequently, on June 21st, 1886, at the request of Mr. James W. Smith, then President of the Consolidated Gas Company, a report outlining the future policy of the Company was presented by Mr. William H. Bradley, then Engineer of the 44th Street Station of the Company, now Chief Engineer, in which the removal of the gas plants from Manhattan Island was strongly advocated. Following this report, the question was thoroughly ventilated in the public press, with strong advocacy of the suggestion that all companies then manufacturing gas on the Island of Manhattan should remove to sites outside the city, and that a great central manufacturing plant should be created on the Flushing Meadows, from which gas should be piped to a convenient point whence it might be conveyed by tunnel under the East River to Manhattan. For several years, however, nothing was done with this proposition. At that time the Ravenswood Gas Company, having a small manufacturing plant at the foot of Webster Avenue, East River, supplied gas locally in Long Island City. About 1891, this company was reorganized by Mr. Emerson McMillin, its corporate title was changed to East River Gas Company, and it obtained from the Legislature of the State of New York the necessary franchise rights to supply gas to New York City by a tunnel to be constructed under the East River.

In 1892 and 1893 the writer's firm constructed, for this new corporation, a small tunnel (internal diameter 10 ft. 2 in. in the por-

tions which were iron lined and a rectangular section 10 by 8 ft. in the portions in solid rock) under the East River, crossing the two channels and Blackwell's Island. This tunnel (shown on Plate XXIV) extends from the foot of East 71st Street, Manhattan, to the Ravenswood works of the Company at the foot of Webster Avenue, Long Island City. Borings and soundings were made at close intervals across the channels, and as far as information could be obtained (with the extremely rapid current running in the East and West Channels of the East River), there was every reason to believe that the construction of a tunnel on that line would be through a practically continuous bed of rock, although the geological information indicated clearly the intrusion of a seam of dolomitic limestone under each channel.

The work on this tunnel was let out by contract, and proceeded very slowly for a considerable time, when, having sunk both shafts and having turned the tunnel on the Manhattan side of the river, decomposed rock was encountered at the contact between the gneiss and dolomite under the West Channel. The contractors immediately abandoned the work, and it was then carried on by the writer's firm, as Constructing Engineers, for account, and on behalf, of the Gas Company.

This tunnel was at a depth of nearly 100 ft. below mean low water. The depth of the water in the West Channel was 65 ft., but that in the East Channel was only about 30 ft. Under the West Channel two comparatively narrow sheared contacts in the dolomite rock were badly decomposed; under the East Channel the contacts between the Fordham gneiss, the mica schist, and the dolomitic formation were also extensively distorted, and, to some extent, decomposed, but the whole of this tunnel was completed successfully with the use of air pressure, the depth not being too great to prohibit this method of construction.

After the completion of this tunnel the East River Gas Company passed into the ownership of the Consolidated Gas Company, and thereafter, the Astoria Light, Heat and Power Company was organized to construct a great central gas manufacturing plant to supply gas to all the constituent companies. This plant was established on a large tract of property at Lawrence Point, Long Island, at the northerly end of the channel known as Hell Gate.

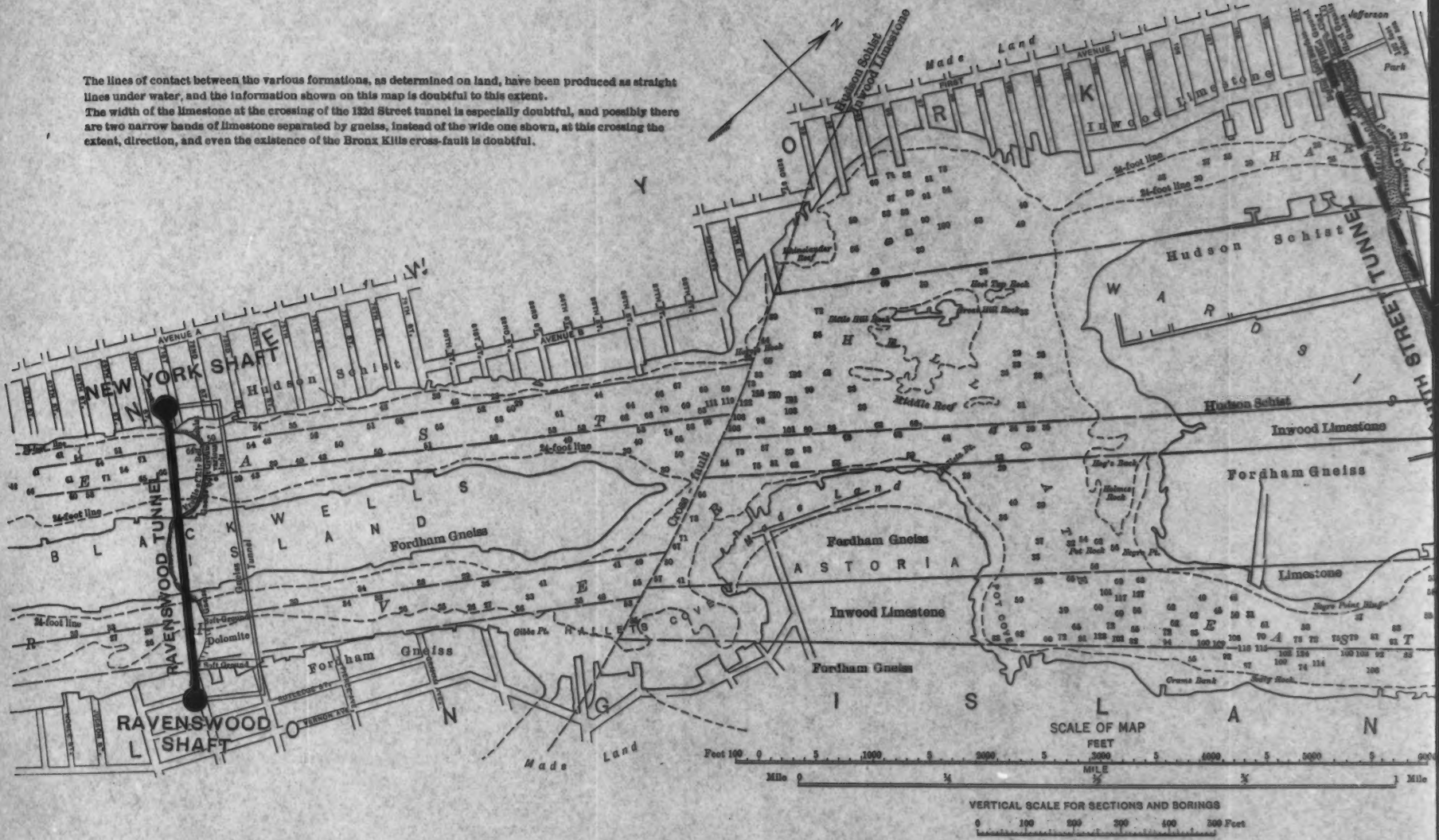
For several years gas from both the Astoria and Ravenswood Plants, for the supply of Manhattan and The Bronx, had been carried through a large pipe line to Ravenswood and thence to Manhattan through the 71st Street tunnel in two 36-in. mains. The supply for The Bronx had been conducted in pipes laid in open coffer-dam construction across the Harlem River, involving a devious detour.

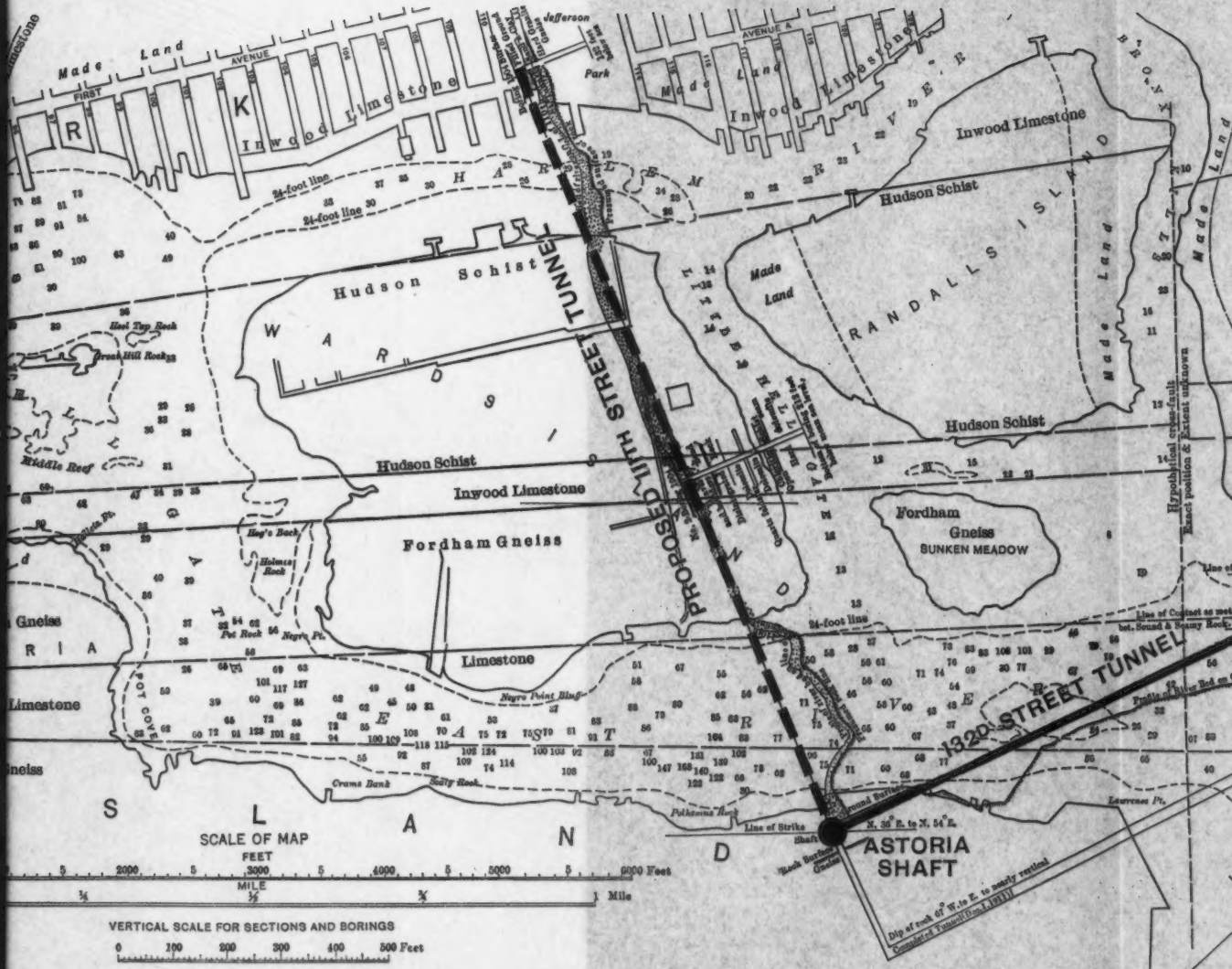
About 1903 it became necessary to consider an independent means of conducting to Manhattan the gas manufactured at the Astoria Plant, as at that time the center of gravity of the gas consumption of the Boroughs of Manhattan and The Bronx combined was at a point south of the Harlem River.

Long legal proceedings, to obtain an easement for a right of way for a tunnel or other means of communication, delayed procedure with this undertaking for several years.

The line of this tunnel was definitely fixed by the necessities of the commercial situation, regardless of geological considerations, as the New York terminus had to be at 111th Street and Pleasant Avenue, where the Gas Company had a distributing station, and the Long Island terminus had to be on the property of the Company at Astoria, just north of Winthrop Avenue. The line, therefore, was laid out between these two points, the Astoria end being kept as close as possible to Winthrop Avenue, in order to conform to the final layout planned for the gas holders of the Company. This line passes under Ward's Island, where it was intended to sink an intermediate shaft for construction purposes, and from this shaft it was contemplated to drive a branch tunnel to a point in The Bronx. Borings were made at the sites of the entrance, outlet and intermediate shafts, and the geological features were studied carefully.

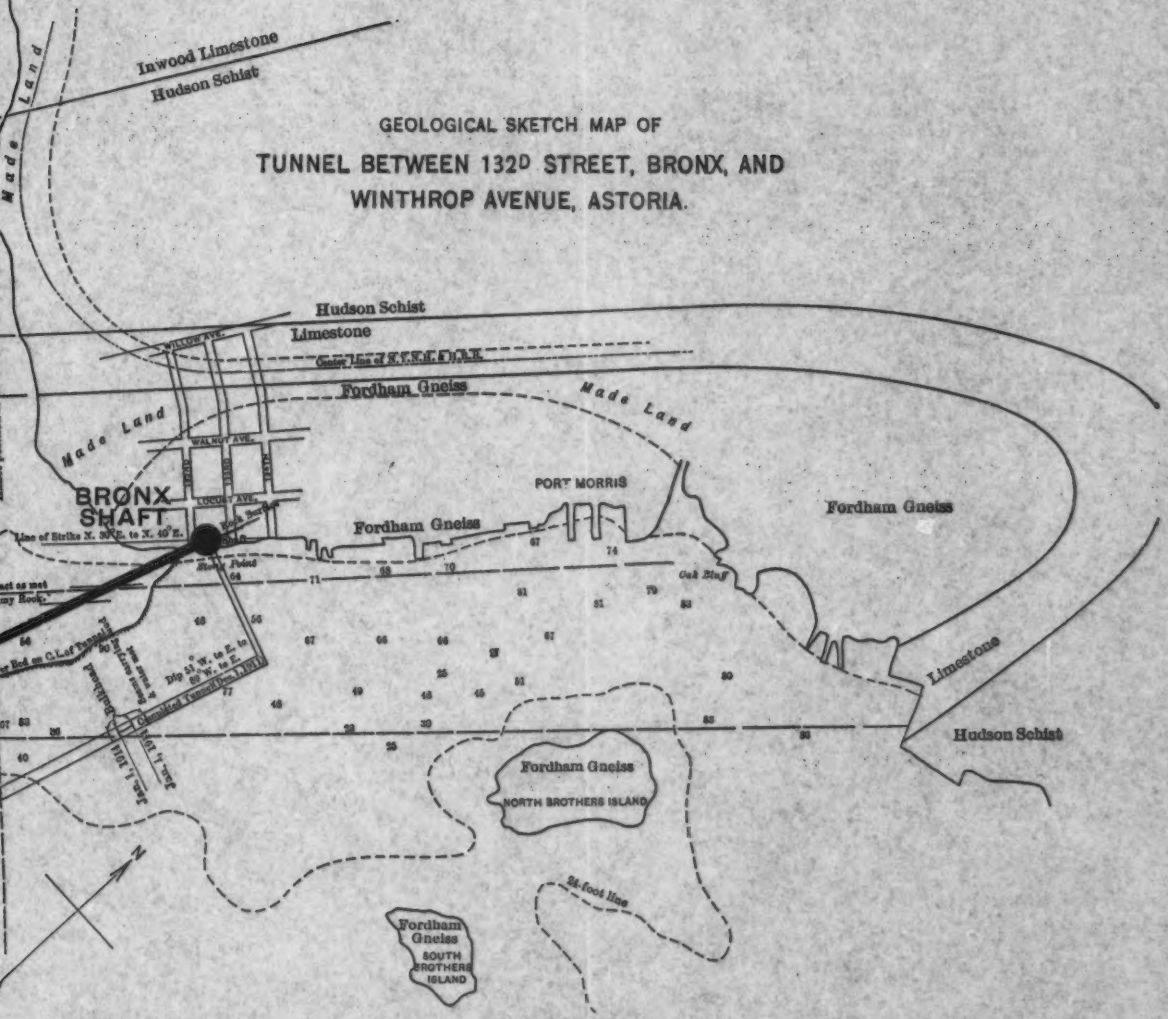
As the depth of the water in the channels on this line was somewhat in excess of 100 ft., and was so great that it was impossible to use air pressure, and as the known geological conditions made it certain that the contacts between the gneiss and dolomite formations would give more or less trouble, it was considered desirable to place the tunnel deep in the rock, in order to minimize trouble from open seams. On the other hand, the necessity (in event of a deeper location) for lengthening vertically the pipes for carrying the gas made it undesirable that these shafts should be deeper than was reasonably


The lines of contact between the various formations, as determined on land, have been produced as straight lines under water, and the information shown on this map is doubtful to this extent.
The width of the limestone at the crossing of the 132d Street tunnel is especially doubtful, and possibly there are two narrow bands of limestone separated by gneiss, instead of the wide one shown, at this crossing the extent, direction, and even the existence of the Bronx Kills cross-fault is doubtful.
N
Hudson Schist
Inwood Limestone
Made Land
NEW YORK SHAFT
RAVENSWOOD SHAFT
RAVENSWOOD TUNNEL
132d STREET TUNNEL
Hudson Schist
Fordham Gneiss
Inwood Limestone
Limestone
Dolomite
Cross Fault
BLACKWELLS ISLAND
ASTORIA
WARD'S ISLAND
SCALE OF MAP
FEET
MILE
VERTICAL SCALE FOR SECTIONS AND BORINGS
0 100 200 300 400 500 Feet
Soundings: The Soundings are expressed in feet, and show the depth at mean low water



rs: The Soundings are expressed in feet, and show the depth at mean low water

PLATE XXIV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXX, No. 1359.
DAVIES ON
THE ASTORIA TUNNEL.



necessary to assure that the tunnel would pass below any serious difficulties of geological contact. Ultimately, the depth for the tunnel was decided on as 186½ ft. below mean sea level at the Manhattan Shaft descending to 218 ft. below that level at the Long Island Shaft. Within a prism on this line and grade, easements for right of way were acquired, and all legal proceedings were completed.

As before stated, these legal proceedings consumed considerable time, and in the interim the center of gravity of gas consumption had moved north of the Harlem River, with every indication that it would extend still farther north; and, by the time the Gas Company was ready to proceed with the work, it had become desirable to discontinue, for the time being, the plan for constructing the tunnel on the original line to 111th Street, and, instead, to proceed with that of delivering gas directly from the Astoria Plant to the Borough of The Bronx. At the same time, the Company did not desire, by any means, to abandon its rights to construct at a later date the tunnel from Astoria to East 111th Street, as originally laid out. This rearrangement necessitated the retention of the shaft near Winthrop Avenue, Astoria, and the purchase of land at Port Morris, at the foot of 132d Street, fronting on the East River, on which to construct a shaft as the northerly outlet of the tunnel, near to, and suitable for a connection with, the distribution system of the Central Union Gas Works, at the foot of 138th Street and Locust Avenue, The Bronx.

The location of the underground prism for the easement for construction of the proposed future tunnel to 111th Street determined the depth of the Astoria Shaft for the tunnel to 132d Street, and an allowance for the necessary drainage gradient toward Astoria determined the limitations of depth at the Bronx Shaft. At the same time, the grade of the tunnel was lowered to the utmost limit permissible within the prism forming the subterranean easement, as it was recognized that on this line the axis of the tunnel would be nearly parallel to the line of strike of the rock, and consequently to the planes of contact between the different geological formations to be penetrated, and therefore, in all probability, would involve more difficulties than the previously projected tunnel to 111th Street, which passed almost at right angles across these contacts. However, the

Gas Company considered that, notwithstanding this, it would be simpler and wiser to face the constructive difficulties than to be under the necessity of adopting a new location which would involve a revival of the entire legal proceedings; and, on this basis, the work hereinafter described was designed and carried out. Plate XXV is a profile of the tunnel and the river bottom.

On account of the uncertainties which might arise in construction, as well as for various other reasons, the Company decided not to invite bids from contractors on this undertaking, but to carry out the work departmentally by its own forces, and the writer's firm was retained as Constructing Engineers in respect to the design and construction.

The following description records for the most part a work of tunnel construction of large magnitude, but comparatively simple in character, excepting only that the general location was at a deep level, with access only at vertical terminal shafts nearly a mile apart, and practically entirely below navigable waterways.

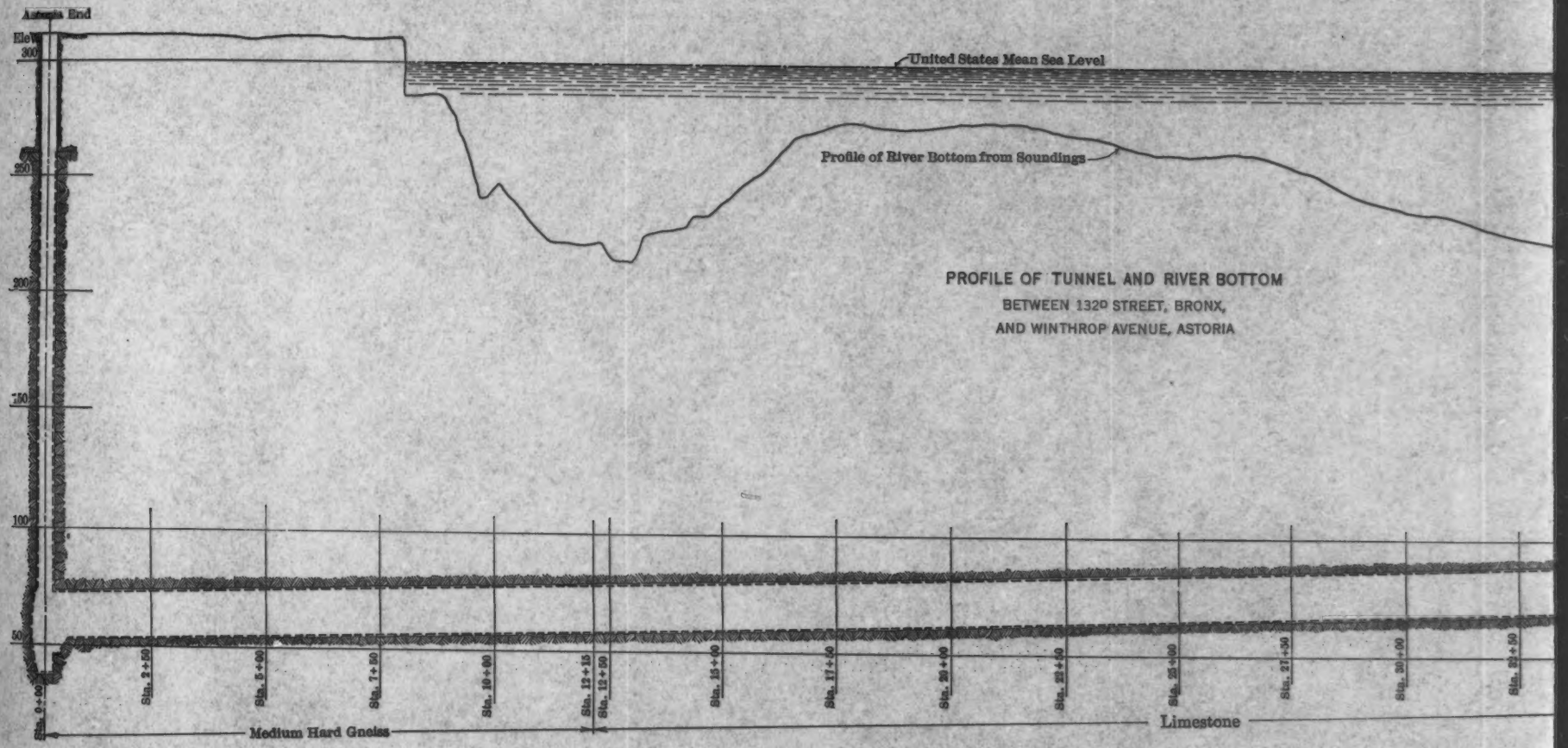
The intention, therefore, in the following presentation, is to describe, in outline only, the work of normal character involved in this construction, and to emphasize more particularly the difficulties encountered, and the methods by which they were overcome.

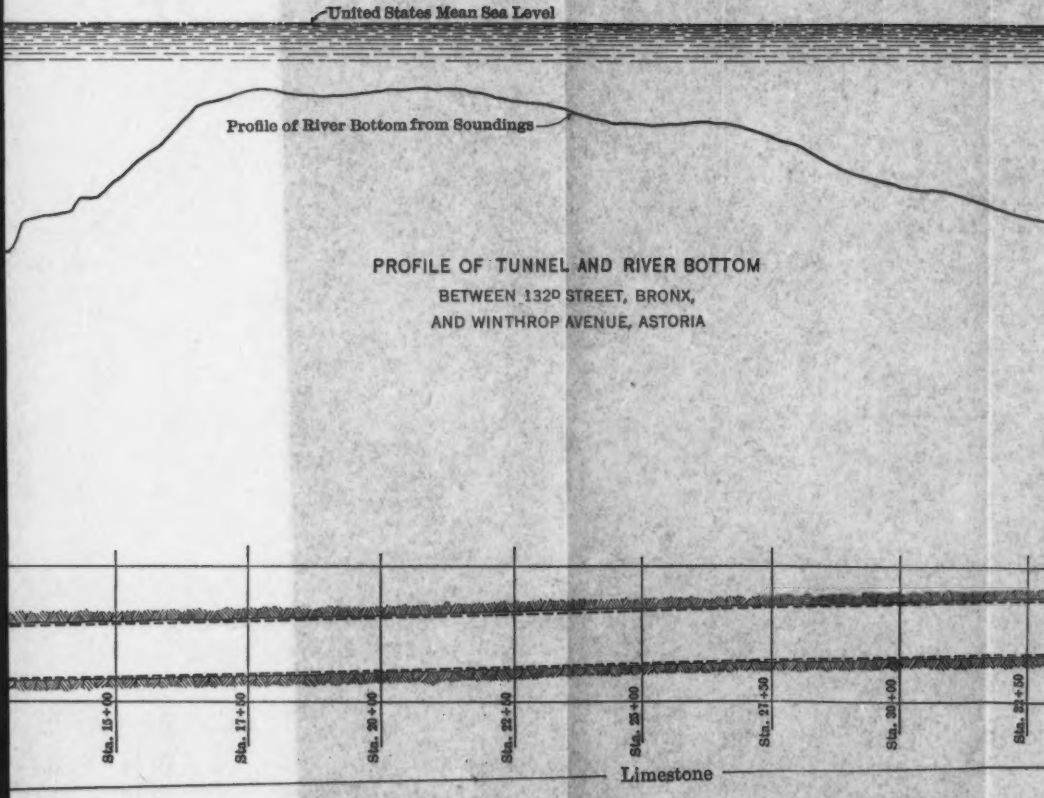
2.—Construction.

The construction involved in this undertaking consisted of a vertical shaft, 277 ft. deep, on the property of the Astoria Light, Heat and Power Company, at Astoria, a vertical shaft, 242 ft. deep, on private property in The Bronx, and a tunnel connecting these shafts.

Anticipating the future construction of the previously planned tunnel direct to 111th Street, Manhattan, the shaft at Astoria was made larger than the one at The Bronx, in order to accommodate the future shaft installation necessary for the two tunnels; and a short portion of the tunnel on the line to 111th Street, Borough of Manhattan, was constructed, so that, as future necessities arose, this tunnel could be extended without disturbing the Astoria-Bronx service.

The necessities for the gas distribution involved the provision of two 72-in. cast-iron mains from Astoria to The Bronx, with space in the upper portion of the tunnel for future developments and utilities.

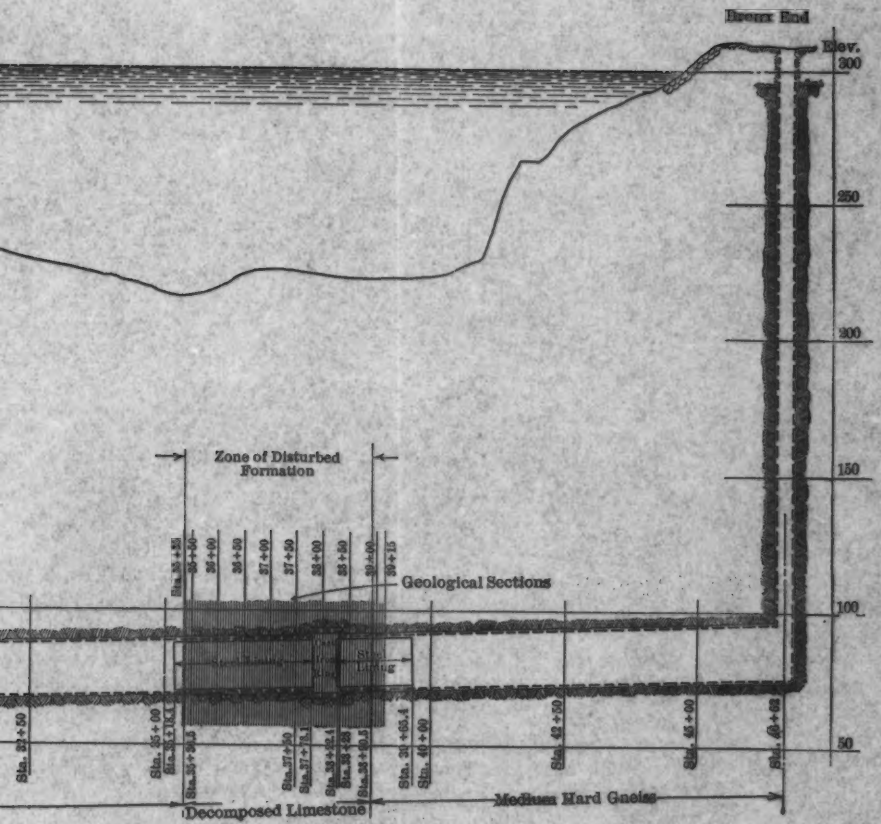

Astoria End
Elev
300
250
200
150
100
50
United States Mean Sea Level
Profile of River Bottom from Soundings
PROFILE OF TUNNEL AND RIVER BOTTOM
BETWEEN 132ᴰ STREET, BRONX,
AND WINTHROP AVENUE, ASTORIA
Sta. 0+00
Sta. 2+50
Sta. 5+00
Sta. 7+50
Sta. 10+00
Sta. 12+15
Sta. 12+50
Sta. 15+00
Sta. 17+50
Sta. 20+00
Sta. 22+50
Sta. 25+00
Sta. 27+50
Sta. 30+00
Sta. 32+50
Medium Hard Gneiss
Limestone



PROFILE OF TUNNEL AND RIVER BOTTOM

BETWEEN 132D STREET, BRONX,

AND WINTHROP AVENUE, ASTORIA


Bronx End
Elev.
300
250
200
150
100
50
Zone of Disturbed Formation
Geological Sections
Sta. 35+55
35+00
36+00
36+50
37+00
37+50
38+00
38+50
39+00
39+15
Sta. 33+50
Sta. 35+00
Sta. 35+30.5
Sta. 37+20
Sta. 37+71.1
Sta. 38+25.4
Sta. 38+38
Sta. 38+40.5
Sta. 39+63.4
Sta. 40+00
Sta. 42+50
Sta. 45+00
Sta. 47+63
Decomposed Limestone
Medium Hard Gneiss

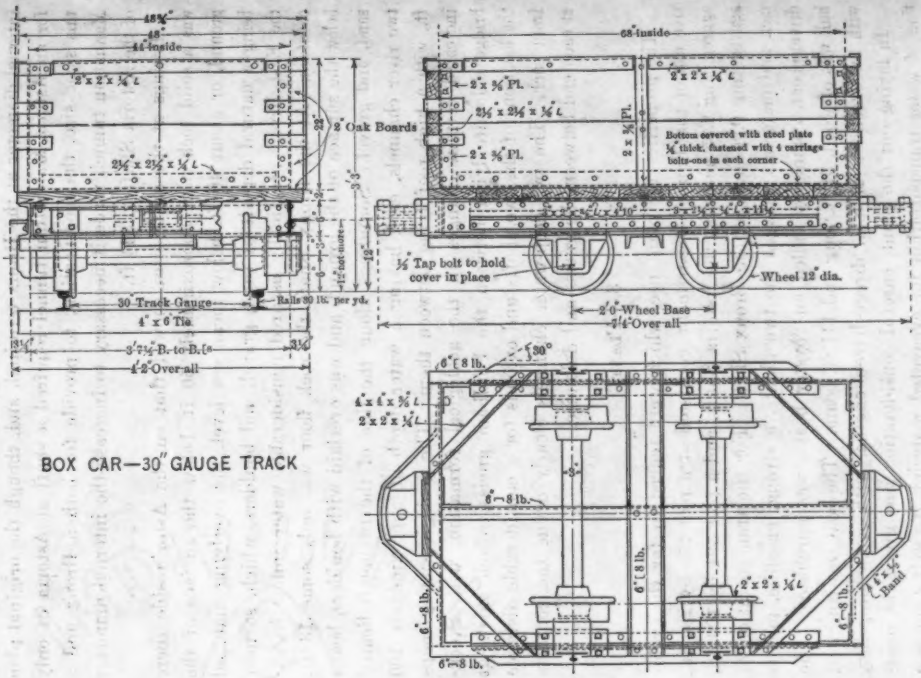
This caused the adoption of a tunnel cross-section 18 ft. on the vertical axis and 19 ft. on the horizontal axis. The vertical pipe risers, elevator frames, etc., made it necessary that the shaft should be 26 ft. in internal diameter at the Bronx terminal, and, though the original plan for an Astoria-Manhattan tunnel required a shaft at Astoria of only this same size, the modification, to provide for both a Bronx and a Manhattan tunnel, made it necessary to increase the internal diameter of the Astoria Shaft to 34½ ft.

Borings at the shaft sites showed that on the Astoria side there was a solid rock floor approximately 50 ft. below the surface of the ground, or about 40 ft. below mean sea level, the overlying material being a natural deposit of sand, gravel, and boulders, which, so near the East River, practically assured considerable water inflow in excavation. At the Bronx Shaft the rock floor was only some 13 ft. below the surface of the ground, and was overlaid with boulders, beach sand, and gravel. Soundings along the line of the tunnel indicated two river channels, each having a water depth of approximately 100 ft., with a high reef lying between them. The geologic survey maps indicated certain changes in the geological formation, with Stockbridge dolomite lying between the Fordham gneiss, which exists at the locations of the two shafts and extends for a considerable distance from each. The two deep-water channels clearly define the contacts at each end between the gneiss and dolomite.

3.—Plant.

The properties for plant use, which were fenced in for the exclusive use of the Tunnel Department, consisted of a plot 255 by 230 ft., aggregating 1.35 acres, at the Astoria Shaft, and a plot 394 by 100 ft., aggregating 0.96 acre, at the Bronx Shaft. For the transportation of men and materials between the two plants, a strong, gasoline, motor-driven boat, commonly called an "oyster boat", was purchased. The hull was of wood, 51 ft. long and 14½ ft. beam. The boat was equipped with an excellent, 25-h.p., direct-driven engine, with screw propeller.

In laying out the plant, careful consideration was given the question of water transportation for excavated material, involving the use of scows and the utilization of the excavated material to fill other lands owned by the Company. A dock existed at the Astoria plant, but there was only a broken rock shore with extremely steep slopes



BOX CAR—30" GAUGE TRACK

FIG. 1.

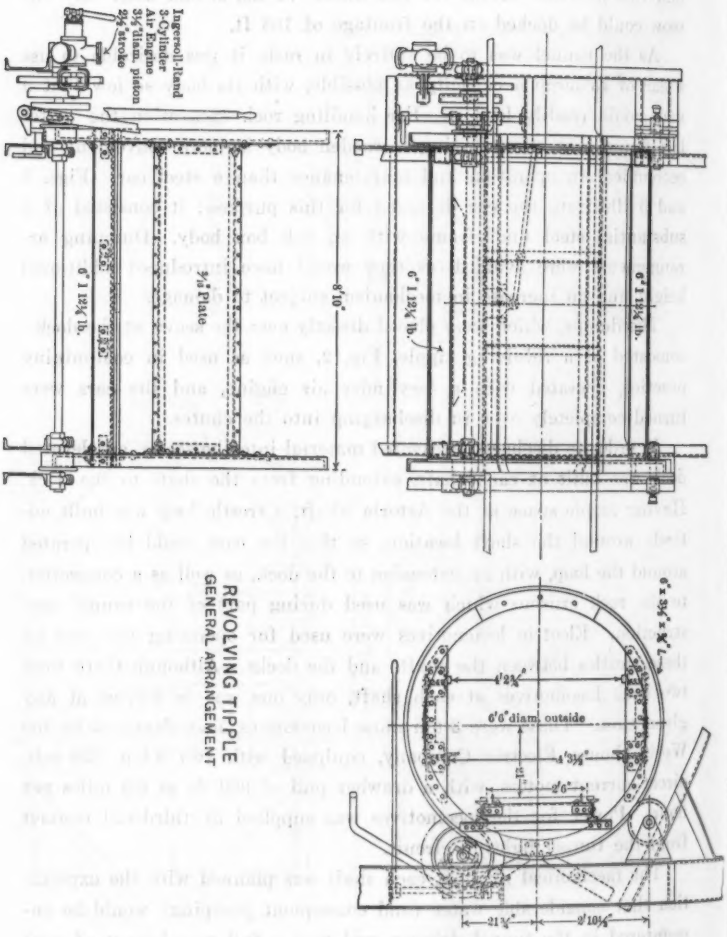
at the Bronx plant, at which site a crib dock had to be built. At the Astoria dock, 230 ft. frontage was allotted to the Tunnel Department, and this provided berths for two scows; at the Bronx dock only one scow could be docked on the frontage of 108 ft.

As the tunnel was to be entirely in rock, it was desirable to use a car of as large a capacity as possible, with its body so low that a man could readily load it. For handling rock excavation the writer has found that a car having a wooden body is more convenient and economical in operation and maintenance than a steel car. Figs. 1 and 3 illustrate the car designed for this purpose; it consisted of a substantial steel underframe with an oak box body. Dumping arrangements were avoided, as they would have introduced additional height and an increase in mechanism subject to damage.

The dumps, which were placed directly over the scows at the docks, consisted of a revolving tipple, Fig. 2, such as used in coal-mining practice, operated with a 3-cylinder air engine, and the cars were turned completely over in discharging into the chutes.

In order to discharge excavated material into the scows, an elevated deck was built at each shaft, extending from the shaft to the dock. Having ample space at the Astoria Shaft, a trestle loop was built entirely around the shaft location, so that the cars could be operated around the loop, with an extension to the dock, as well as a connection to the rock crusher which was used during part of the tunnel construction. Electric locomotives were used for operating the cars on these trestles between the shafts and the docks. Although there were two such locomotives at each shaft, only one was in service at any given time. These were 3-ton mine locomotives, manufactured by the Westinghouse Electric Company, equipped with two 8-h.p. 250-volt, direct-current motors, with a drawbar pull of 900 lb. at 6.6 miles per hour. Power for the locomotives was supplied by third-rail contact from the tunnel lighting circuit.

The mechanical plant at each shaft was planned with the expectation that considerable water (and consequent pumping) would be encountered in the tunnel driving, and that a full complement of rock drills and labor-saving devices would be necessary, and with the intention of constructing half the length of the tunnel from each shaft. The following plant was used.



REVOLVING TIPPLE
GENERAL ARRANGEMENT

FIG. 2.

ASTORIA PLANT.

Boiler-Room.—

Four 302-h.p. Heine water-tube boilers, } 1 840 total boiler h.p.
Two 316-h.p. " " " " }

Six 20-in. Typhoon turbine blowers—L. J. Wing Manufacturing Company.

Engine-Room.—

Two 100-lb. pressure, Ingersoll-Rand air compressors; total capacity, 3 200 cu. ft. per min.

Two 42-in. Ingersoll-Rand after-coolers.

Two 54 by 120-in. Ingersoll-Rand air receivers.

One hot-well tank, 48 in. diameter, 96 in. high—G. Stuebner.

Two 50-kw., 200-ampere, 250-volt, General Electric generators.

Two 75-h.p. Harrisburg Foundry and Machine Company, horizontal engines.

One Walker Electric Company switch-board for 250-kw. generators.

Two 1 200-boiler h.p. Worthington boiler feed pumps.

One 250-gal. per min. Worthington duplex pump.

One surface condenser and feed-water heater combined—12 600 lb. of steam per hour—Wheeler Condenser and Engineering Company.

One vacuum oil separator and simplex pump—12 000 lb. steam per hour—Warren Webster Company and Warren Steam Pump Company.

Two closed water-tube feed-water heaters—800 h.p. to raise 24 000 lb. of water per hour from 60 to 200° Fahr.—F. L. Patterson and Company.

One 30-gal. oil filter—Hall Manufacturing Company.

Surface.—

One 45-ft. derrick.

Two 50-ft. derricks.

Three Lidgerwood Manufacturing Company hoisting engines for derricks.

One 1-cu. yd. Smith concrete mixer and engine.

One No. 4 gyratory stone crusher—Allis-Chalmers Company.

One 8 by 12-in. Buckeye vertical engine.

One 42-in. Ingersoll-Rand air reheater.

Two 3-ton electric mine locomotives—Westinghouse Electric and Baldwin Company.

Blacksmith-Shop.—

One Ingersoll-Rand Ajax drill sharpener.

One Ingersoll-Rand furnace.

Two 36 by 36-in. air forges—Buffalo Forge Company.

Two hand forges—Buffalo Forge Company.

Machine-Shop.—

One 18-in. horizontal lathe—Niles Bement Pond Company.

One 16-in. shaper " " " "

One 24-in. drill press " " " "

One ¾ to 1½-in. bolt-threading machine—Wells Brothers.

One 2½ to 8-in. pipe-threading machine—Curtis and Curtis.

One power hack-saw—Frevert Machine Company.

One 12-in. emery wheel—Niles Bement Pond Company.

One 6 by 6-in. vertical engine—B. F. Sturtevant.

Carpenter-Shop.—

One 16 to 20-in. wood-frame rip-saw, 7½ to 15 h.p.—American Woodworking Machine Company.

One swing-saw.

One band-saw, 1-in. saw, 33-in. band wheels—H. B. Smith and Company.

Two Fairbanks grindstones.

One 6 by 6-in. vertical engine—B. F. Sturtevant.

Shaft Hoisting Equipment.—

One 10 by 12-in. reversible, compound-geared, elevator engine—Lidgerwood Manufacturing Company.

One 12¼ by 15-in. reversible, link-motion, single-geared, elevator engine—Lidgerwood Manufacturing Company.

One 66 by 72-in. passenger elevator cage.

Two 66 by 96-in. material elevator cages.

Four 1½-yd. shaft buckets—G. Stuebner.

Three ¾-yd. tip buckets—G. Stuebner.

BRONX PLANT.

Boiler-Room.—

Three 375-h.p., Babcock and Wilcox boilers—1 125 total boiler h.p.

One induced-draft fan and engine.

Engine-Room.—

Two 100-lb. pressure, Ingersoll-Rand air compressors; total capacity, 3 200 cu. ft. per min.

Two 42-in. Ingersoll-Rand after-coolers.

Two 54 by 120-in. Ingersoll-Rand air receivers.

One hot-well tank, 48-in. diameter, 96 in. high—G. Stuebner.

Two 50-kw., 200-ampere, 250-volt, General Electric generators.

Two 75-h.p. Harrisburg Foundry and Machine Company, horizontal engines.

One Walker Electric Company switch-board for 250-kw. generators.

Two 1 200-boiler h.p. Worthington boiler feed pumps.

One 250-gal. per min. Worthington duplex pump.

One surface condenser and feed-water heater combined—12 600 lb. of steam per hour—Wheeler Condenser and Engineering Company.

One vacuum oil separator and simplex pump—12 000 lb. steam per hour—Warren Webster Company and Warren Steam Pump Company.

One 30-gal. oil filter.

Surface.—

Two 50-ft. derricks.

Two 8¼ by 10-in. non-reversible engines for derrick—Lidgerwood Manufacturing Company.

One 1-cu. yd. Smith concrete mixer and engine.

One 42-in. Ingersoll-Rand air reheater.

One 60-h.p. vertical boiler—Wickes Brothers.

Two 3-ton electric mine locomotives—Westinghouse and Baldwin Companies.

Shaft Hoisting Equipment.—

One 10 by 12-in., reversible, compound-geared, elevator engine—Lidgerwood Manufacturing Company.

One 12¼ by 15-in., reversible, link-motion, single-geared elevator engine—Lidgerwood Manufacturing Company.

One 66 by 72-in. passenger elevator cage.

Two 66 by 96-in. material elevator cages.

Four 1½-yd. shaft buckets—G. Stuebner.

Three ¾-yd. tip buckets—G. Stuebner.

Tunnel Equipment.

Rock-Drilling Equipment.—

Six Ingersoll-Rand rock drills, 2¼-in. pistons.
Thirty Ingersoll-Rand rock drills, 3¼-in. pistons.
Six Ingersoll-Rand rock drills, 3⅜-in. pistons.
Twenty-three Ingersoll-Rand hand hammer drills.
Sixteen Ingersoll-Rand 5½ by 96-in. drill columns.
Twenty-eight Ingersoll-Rand drill tripods.

Concrete Equipment.—

Two Blaw steel arch and side-wall forms.
Two " " side-wall forms.
Twelve bottom-dump buckets—G. Stuebner.

Grouting Equipment.—

Two 200-lb. pressure, Ingersoll-Rand, straight-line, air compressors.
One 500-lb. pressure, Ingersoll-Rand, straight-line, air compressors.
Two 36 by 96-in. air receivers—Logan Iron Works.

} Total capacity 900 cu. ft. per min. In Engine-room.

Two 500-lb. pressure, cast-steel, grout pans, without engines.
Eight 200-lb. pressure, grout machines and engines—Cockburn Barrow and Machine Company.

Pumping Plant.—

Two	Cameron	piston	pumps,	115	gal.	per	min.
One	"	sinking	"	200	"	"	"
Five	"	piston	"	330	"	"	"
Nineteen	"	"	"	600	"	"	"
One	"	"	"	800	"	"	"
One	"	"	"	1 000	"	"	"

Rolling Stock and Haulage.—

Two 8 by 10-in. endless-cable, haulage engines—Lidgerwood Manufacturing Company.
Four 5 by 6-in. single-drum, hoisting engines—Lidgerwood Manufacturing Company.
One hundred 1½-yd. wooden mine cars.
Thirty "V" steel, side-dump cars.

Ventilating Plant.—

Two 12-in. Sirocco fans with motors—American Blower Company.

One 30-in. ventilating fan and engine—B. F. Sturtevant.

One 42-in. centrifugal blower and engine—B. F. Sturtevant.

4.—Shaft Sinking.

Shaft Sinking in Earth.

Astoria Shaft.—The sinking of the Astoria Shaft began on September 12th, 1910. Steel sheet-piling of United States Steel Company section was used for the exterior sheeting through the earth section, for a depth of 52 ft., and was braced with 16-sided timber frames, 4 ft. apart, strapped at the joints with steel plates, to which were attached turnbuckle tie-rods extending across the shaft for use in maintaining the circular form. (Fig. 4.) The dump trestle having been constructed to circular form at a height of 15 ft. above the ground, and the excavation being removed to 8 ft. below the ground, a frame was prepared to the correct circular form, plumbed accurately below the trestle form, and served as a guide for driving the sheet-piling. The piling was cut in two lengths for the total depth, with joints staggered, and was driven with an Arnott 2 000-lb. steam pile-hammer suspended from a derrick.

Excavation was removed as the sheet-piling was driven, in order to avoid any distortion by over-driving against boulders or rock.

Having sunk the shaft to rock level, excavation was carried down into the rock about 4 ft. below the point of the piling, and concrete was placed to half the ultimate thickness for the full depth of the excavation, in order to secure the sheet-piling and timber frames from injury during the further extension of the shaft construction.

Bronx Shaft.—At the Bronx Shaft, the shallow depth of the overlying soil and material, as well as its character, made it unnecessary to use steel sheet-piling. Wooden sheeting of 4-in. tongued and grooved stuff, supported by 12-sided timber frames, 4 ft. apart, was used through the 13 ft. of earth to rock level; the permanent lining inside this sheeting was concreted on reaching the rock floor.

Shaft Sinking in Rock.

Shaft sinking in rock was carried out by the ordinary methods, the usual practice being to line up with concrete every 30 ft., as the

rock was of treacherous character, and the introduction of the lining insured the employees against accidents.

For hoisting and removing the rock excavated from the shafts, and to avoid the danger of using a swinging bucket operated from the boom of a derrick, the permanent head-frame, Fig. 5, to be equipped later for the use of elevator cages, was erected, with overhead hoisting sheaves and the regular elevator hoisting engines. One of these elevator wells was then fitted with guide-framing below the landing, and at the level of the landing a broad-gauge track crossed the elevator opening, with trap-doors fitted to insure safety. The spoil buckets were hoisted through this trap and landed on a flatcar run through the elevator opening before the hoisting rope was detached.

Water entered from numerous rock joints at various points in sinking the shafts, but not in such volume as to retard the progress of the work, except by the time lost in the removal by the derrick of the pumps and hose when blasting. This shaft water was readily handled in each shaft by one 350-gal. per min. steam-driven pump.

Two water rings were placed in each shaft as the sinking proceeded, in order to intercept the water seepage and prevent it from pouring on the men working below. Each ring was simply a 6-in. wooden shelf with gutter board sheathed with zinc, and a 3-in. pipe extending to the shaft bottom for drainage. Later, these leaks were stopped by grouting, while the driving of the tunnel was in progress.

Astoria Shaft.—At the bottom of the Astoria Shaft the design provided for an enlargement, about 41 ft. high, 34½ ft. wide, and extending some 19 ft. beyond the neat shaft lines. This was to be used ultimately as an operating chamber for the distribution of gas, but it, as well as the short length of 100 ft. of tunnel in the direction of the future Manhattan Shaft at 111th Street, was utilized, during the construction of the tunnel, as a pump chamber and store-room. This enlargement extended 16 ft. below the tunnel invert, and, in addition, contained a 5-ft. circular sump 8 ft. deep, so that ample provision was afforded for the accumulation of the expected water flows and for settling any solid matter carried with the water.

Bronx Shaft Sump.—As the tunnel drained southward toward Astoria, a similar sump was not required in the Bronx Shaft, but ample precautions were taken by the construction of a rectangular sump, 21 ft. long, 12 ft. wide, and 8½ ft. deep.

FIG. 3.—BOX CAR FOR EXCAVATION FROM ASTORIA TUNNEL.

FIG. 4.—LOOKING DOWN THE ASTORIA SHAFT.

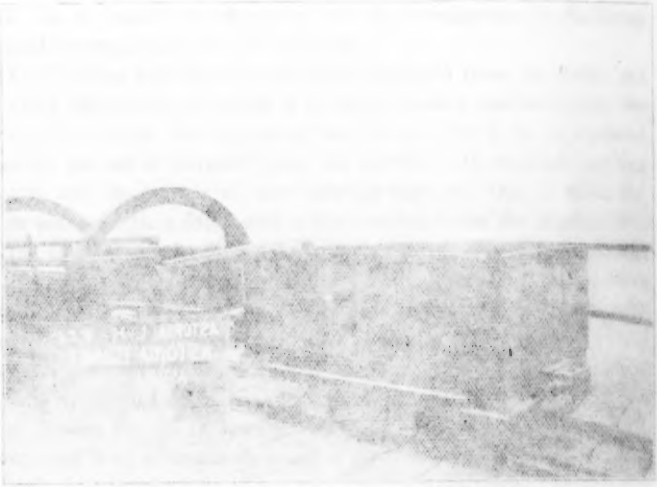


FIG. 5.—HEAD-HOUSE, ASTORIA END OF TUNNEL.

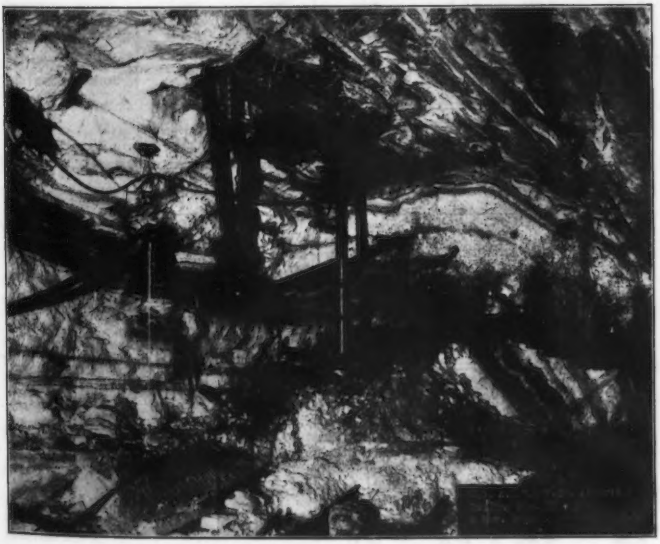


FIG. 6.—HEADING AT STATION 35 + 36.

Quantities.—The following tabulation presents the unit quantities relating to the shaft sinking in rock:

Drilling and Blasting.	Astoria Shaft.	Bronx Shaft.
Feet drilled per actual cubic yard.....	3.9	3.3
Linear feet of drill steel used per cubic yard..	0.174	0.172
Pounds of 60% Forcite used per cubic yard....	1.2	1.1

Progress and Unit Costs.—Excellent progress was attained in the shaft construction at both ends. This, together with the unit costs of shaft construction, is shown in Table 1. The dimensions of the shaft are given in Table 2. The total unit costs of the shafts are given in Table 3.

TABLE 1.—UNIT COSTS OF SHAFT CONSTRUCTION.

	Actual Cubic Yards.		Working Days of 24 Hours Each.		Daily Average, in Actual Cubic Yards.	
	Astoria Shaft.	Bronx Shaft.	Astoria Shaft.	Bronx Shaft.	Astoria Shaft.	Bronx Shaft.
Excavating earth...	2 687	367	16.8	2.9	159.7	126.6
Excavating rock....	11 542	5 939	123.4	97.9	93.5	60 7
Placing concrete....	3 178	1 621	37.2	23.3	85.5	69.6
Total construction..	.....	.....	177.4	124.1	.....	.....

TABLE 2.—DIMENSIONS OF SHAFTS.

	Astoria Shaft.	Bronx Shaft.
External diameter, earth section.....	42 ft.	30 ft.
External diameter, rock section.....	38½ ft.	28½ ft.
Internal diameter.....	34½ ft.	26 ft.
Depth.....	276½ ft.	242 ft.

TABLE 3.—UNIT TOTAL COSTS OF SHAFTS.

	Per Actual Cubic Yard.		Per Linear Foot.	
	Astoria Shaft.	Bronx Shaft.	Astoria Shaft.	Bronx Shaft.
Earth excavation.....	$7.05	$2.67	$365.81	$75.39
Rock excavation.....	10.43	11.22	535.84	275.44
Concrete lining.....	14.21	13.76	163.28	92.16

5.—Tunnel Driving Under Normal Conditions.

Bronx Heading.—The Bronx Shaft reached a sufficient depth for turning the heading on February 12th, 1911, and the latter was driven for a short length before completing the shaft and sump, so that tunnel driving really began on March 8th, 1911, and continued under normal conditions through sound, close-grained gneiss until August 29th, 1911. At that time the first appreciable delay occurred when the heading entered the expected contact seam of decomposed, water-bearing dolomite, 771 ft. from the shaft center, at which point the water depth in the river is 80 ft. below mean sea level, and the deep river channel corresponds in location to this dolomitic intrusion.

Astoria Heading.—At the Astoria end, work on the main tunnel did not begin until April 8th, 1911, when the headings of both the Bronx and Manhattan Tunnels were already turned. As in The Bronx, the headings were turned when the shaft reached the level of the floor of the top heading, and prior to completing the shaft sinking; the advance toward The Bronx continued until April 23d, 1911, during which time about 80 ft. of the Manhattan Branch Tunnel were driven. Owing to the large chamber arrangements necessary at the bottom of the Astoria Shaft, work on the main tunnel was not resumed from this end until June 23d, 1911, and it continued without incident until November 7th, 1912, a distance of 3 536 ft. from the shaft center.

Geology.—For the first 1 200 ft. from Astoria the tunnel was driven through a hard, compact, and tough granitic gneiss, requiring heavy drilling and large consumption of powder. For the next 2 336 ft. the tunnel passed through the dolomite, and the passage through the easterly contact between the gneiss and the dolomite was made without any apparent disturbance or shear of the geological structure, a condition quite different from the contact encountered in the Bronx heading, where the geological change was featured by violent shear, accompanied by innumerable water fissures and excessive disintegration. At a point 3 536 ft. from the Astoria Shaft, this heading met the first indications of the water-bearing disintegration of the westerly contact between the gneiss and the dolomite. (Fig. 6.) The final results developed the fact that the entire width of this decomposed contact normal to the strike was some 150 ft., whereas the angle of crossing made

by the tunnel involved a total distance of 350 ft., measured on the axis of the tunnel, or 450 ft. between points of contact on the two side lines.

Excavation.—For the tunnel excavation, the top-heading-and-bench method was adopted, maintaining the bench some 50 ft. behind the heading. During the early part of the work the arch lining was not placed as the excavation advanced, these exposed sections being concreted later by using steel forms, and work was performed simultaneously with, and without hindrance to, the driving of the heading and bench. On deciding to place the arch lining along with the tunnel advance, a system of alternate excavating and concreting was adopted, with excellent results as to progress attained. This method consisted of excavating both the heading and bench from 8 A. M. Monday to 8 P. M. the following Friday, the remainder of the 6-day week (until 8 A. M. Sunday) being occupied in placing the concrete arch. By this method the maximum monthly progress was made, being 269 ft. of full tunnel section excavated and arch lined. The average progress maintained by this method was 53 ft. of heading, bench, and concrete arch per 6-day week, which, in volumetric measure, equalled 675 cu. yd. of neat-line excavation and 81 cu. yd. of neat-line concrete arch.

The average unit drilling and blasting quantities incidental to driving the tunnel 4 274 lin. ft., or 92% of the total length, under normal conditions, are presented in Table 4.

TABLE 4.—AVERAGE UNIT DRILLING AND BLASTING QUANTITIES FOR TUNNEL.

	AVERAGE PER ACTUAL CUBIC YARD.					
	Fordham Gneiss.			Stockbridge Dolomite.		
	Heading.	Bench.	Total.	Heading.	Bench.	Total.
Drilling, in feet	5.64	3.04	4.22	5.09	2.03	3.42
Pounds of powder used	4.20	1.35	2.65	4.35	1.08	2.57
Number of exploders	0.79	0.38	0.57	0.71	0.26	0.46

Overbreakage.—Notwithstanding numerous experiments in the placing of drill holes for face of heading and bench, it was found that neither the gneiss nor the dolomite rock could be blasted without great

irregularity in cross-section. The large quantities of blasting gelatine needed to pull the rock caused considerable overbreakage. Careful and repeated cross-section measurements, checked by the volume of concrete used to complete the lining, indicated that, notwithstanding the particular care taken, overbreakage outside of the neat line of minimum lining section in the sound sections of good rock amounted to 18% in the gneiss and 12½% in the dolomite.

Drilling and Firing.—The typical arrangement and length of drill holes for blasting, for both heading and bench, are given in the diagram, Fig. 7, as applicable to both the gneiss and dolomite, under normal conditions. In all cases holes were started with 3-in. drills, which were reduced ¼-in. in diameter with each 2-ft. advance, to a final diameter of 1¾-in. for the 12-ft. holes.

Fig. 7 also indicates graphically the arrangement of wiring, and the order of shooting, the holes.

Two double-strength "Victor" fuses were always used in regular course to insure the proper explosion of charges.

All firing was done by magneto batteries maintained about 500 ft. back of the heading, the wire connections to the heading always being strung on the side of the tunnel opposite that on which electric light wires were placed.

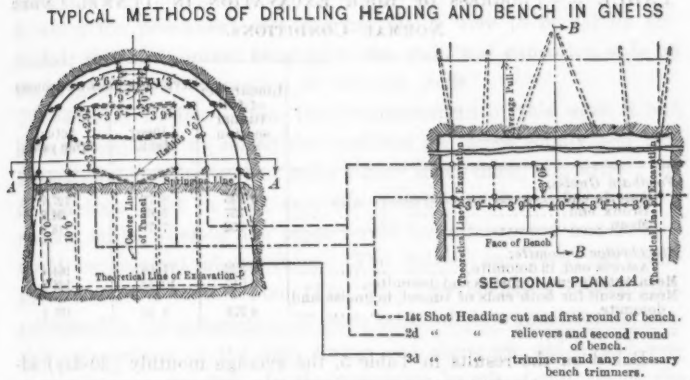
Both for heading and bench excavation ⅜-in. steel shoveling plates were used on the floors for facilitating the mucking operations.

In the varied classes of rock throughout the tunnel headings the speed of drilling was 9.9 lin. ft. per hour per percussion machine. The bench drilling was easily maintained at the equivalent rate of progress.

Practically all the best known brands of drill steel furnished in the New York market were tried out on the work, resulting in the adoption of "ABC non-tempering" steel, which, in these rocks and under the conditions, was found more durable and showed less breakage in dressing and drilling than the others.

The average progress of excavation attained during the tunnel driving under normal conditions, consisting of 4 274 lin. ft., or 92% of the ultimate length, and, based on the advance of one heading only, is given in Table 5. In this tabulation only the total accumulative time chargeable to excavation is considered, with a 24-hour actual working day as the unit.

TYPICAL METHODS OF DRILLING HEADING AND BENCH IN GNEISS



Astoria End	Period from June 22d, 1911, to January 24th, 1912. From Sta. 1+00.7 to Sta. 14+44.8, or 1344.1 ft. Gneiss = 1100 ft. ± Limestone = 244 ft. ±
Bronx End	Period from April 19th, 1911, to July 22d, 1911 From Sta. 46+66.5 to Sta. +39.50.5, or 616 ft. Gneiss = 616 ft.

Item	Astoria Heading	Bronx Heading	Astoria Bench	Bronx Bench
Strength and Size of Powder used	60% 1½-in. Forcite			
Quantity of Powder per Hole	Cut 6 Sticks Reliever 5 " Trimming 4 "		5 Sticks	
Size of Starting and Ending Bits	3-in and 1¾-in.			
Drills Used	E-44 Ing.Rand-3½-in. cylinders. Mounted on Columns. E-94 Ing.Rand-4½-in. cylinders. Mounted on Columns.		E-44 Ing.Rand. 3½-in. cylinders. Mounted on Tripods	
Average number of Drills used at one time	5		4	
Average time drilling a Complete Round	44.1 man-hr.	56.5 man-hr.		
Average elapsed time for removal of Round	15.1 hr.	21.3 hr.		
Average quantity of Powder per cubic yard	4.9 lb.		1.25 lb.	
Average feet of hole drilled per cubic yard	5.2 ft.	5.7 ft.		
Average feet of hole drilled per hour	5.3 ft. per man includes setting up.	4.6 ft. per man includes setting up.		

In Driving through Dolomitic Formation, the Heading Holes were all lengthened 1 ft.

FIG. 7.

TABLE 5.—PROGRESS OF ROCK EXCAVATION IN TUNNEL UNDER NORMAL CONDITIONS.

	Linear feet of full tunnel section.	AVERAGE DAILY PROGRESS.	
		Linear feet.	Actual cubic yards.
Fordham Gneiss:			
Astoria end	1 195	8.72	127.7
Bronx end	757	6.62	94.9
Mean	1 952	7.79	113.0
Stockbridge Dolomite:			
Astoria end, in dolomite	2 322	11.68	164.4
Mean Astoria end, in gneiss and dolomite	3 517	10.43	148.8
Mean result for both ends of tunnel, in gneiss and dolomite	4 274	9.46	135.1

Based on the results in Table 5, the average monthly (30-day) advance of completed tunnel, excavation only, was 284 lin. ft., or 4 054 actual cu. yd., in one heading.

Although the rate of progress was not as great as in other tunnels, of which reports have been published, the results obtained are thought to be very satisfactory, considering the general conditions of hoisting from a deep level, particularly as at no time was it considered by the management desirable to enter into the system of bonusing men to obtain record results. The entire work was carried out on a normal wage basis, without any attempt to offer other or special inducements to the labor to accomplish records. The progress of the work was perfectly regular and remarkably uniform, both at the Bronx and Astoria Shafts, up to the time of meeting the contact of decomposed rock at each end.

Concrete Arch.—The portion of sound rock tunnel driven from both the Bronx and Astoria Shafts was a hard, solid structure, but the rock indicated extensive jointing, with a tendency to scale away and for blocks of rock to fall without warning, a condition due to the character of the rock and also to the heavy blasting operations. Therefore what had been anticipated to be an untimbered tunnel, necessitated early the consideration either of timbering or of carrying the lining along with the tunnel operations, in order to insure the safety of the workmen. After careful consideration of the advantages and disadvantages of these two methods, it was decided to construct the arch lining, and, on account of the original cross-section adopted, to

finish this arch lining to a skewback intended for the bottom concrete in which the permanent cast-iron gas mains were to be solidly embedded; this arch lining, hanging to the arch, was supported only on the irregularly jointed section of the rock walls.

Change of Design.—From the commencement of this work it had been contemplated to embed the cast-iron gas pipes solidly and permanently in concrete, and to make a floor above them, but, after 2 867 ft. had been lined in this way, the design was reconsidered and a revision made, whereby the pipes would be left exposed and a deck would be inserted above them. The tunnel has been left completed according to this plan (Fig. 8). This obviously necessitated, subsequent to the completion of other parts of the tunnel lining, cutting out portions of the arch lining and skewback which had previously been constructed. In the remaining length of the tunnel, the arch concrete was supported by timber sills on posts, the side-walls and invert not being placed until the complete excavation of the tunnel from end to end, when the sills and posts were removed as the wall lining proceeded.

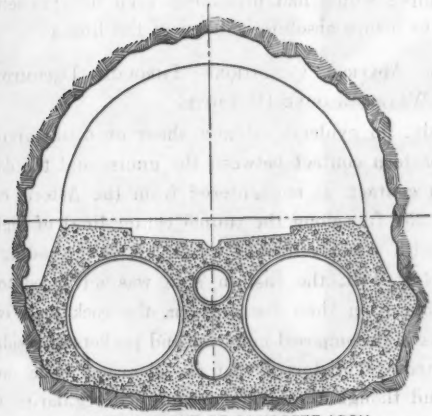
This method of procedure was peculiar, in that, for the entire job, the arch lining was first executed. Then, when the tunnel was holed through, and the steel lining, hereinafter described, was being erected in the decomposed section, the invert was put in place by trimming the rock floor, in lengths ahead of concreting and laying the invert, to accurately placed side-forms. The forward end of each length of invert was laid out with a curved cross-form which gave accurately the section of the concrete to be laid; this was swept with straight-edges, using the last completed length of invert for the rear sweep and a wooden form for the forward end. The joints at the side-forms, as well as the cross-joints, were designed so as to lock thoroughly with the abutting concrete to be placed later. On the completed invert, after it had come to strength, was laid a track of 60-in. gauge, on which were operated two complete collapsible steel forms, each 50 ft. long; and these were used for the construction of the intervening side-walls, the two side-walls being concreted simultaneously. These forms, Fig. 9, were set up 100 ft. apart, and were advanced 100 ft. at each move, so that each form was used for alternate lengths of side-wall lining, thus permitting the setting up of one form while the concreting was proceeding in the other; this enabled continuous working operations.

Although these forms ran on the 60-in. track laid on the tunnel invert, they had on the bottom frame a standard tunnel-gauge track, with a short incline at each end, so that other operations in the tunnel could proceed by running the tunnel cars through the lower portion of the steel frame. There was an elevator at each end of each form to raise the concrete cars to the upper level, so that they could be run to any point for dumping. The collapsible feature of the steel forms was obtained by using jacks and turnbuckles, and the upper platform of the forms, which was the dumping platform, was practically at the springing line of the arch where the joint was made between the side-walls and the arch already in place.

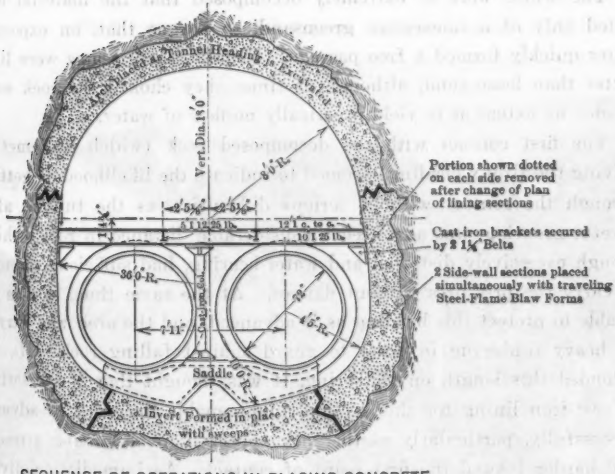
Before placing the forms the surfaces were well oiled. The moving was done by a small air-driven engine on the dumping platform. This engine was also used to operate the elevator. These forms worked admirably after they had been reconstructed, to some extent, to meet the conditions. The progress obtained with their use was one 50-ft. section each working day, and this rate was maintained with almost perfect regularity for a length of 3 310 ft. of tunnel.

The proper keying of the side-wall concrete to the arch skewback was accomplished, after a number of experimental methods were tested, by forming a wooden chute at the base of the joint and to a level of about 1 ft. above it, through which wet concrete was filled in and rammed into the joint with rods, in order to work the concrete into close contact with the arch skewback and enable the air bubbles to escape. This left a projecting lump of concrete at the joint, but, just prior to moving the forms forward, these chutes were removed and the projecting concrete was chipped off to the neat lines while the concrete was still green. The results of this method were quite satisfactory.

Throughout the work of placing the concrete lining in the arch, side-walls, and inverts, it was considered, owing to the heavy pressures which would come on the lining by any seepage of water, that all voids between the arch section and the solid rock should be entirely filled with concrete. Plums of broken stone, however, were used freely where the concrete exceeded in thickness the neat section. Consequently, no loose packing of rock behind the arch or walls was done in any portion of the work, and, throughout the tunnel, on the com-



ARRANGEMENT OF CONCRETE ARCH
FOR ORIGINAL CROSS-SECTION



SEQUENCE OF OPERATIONS IN PLACING CONCRETE
LINING FOR FINAL CROSS-SECTION

FIG. 8.

pletion of the entire lining from end to end, grouting was carried out by the use of grout pipes which had previously been left projecting from the lining so as to insure absolute solidity of the lining.

6.—Tunneling Under Adverse Conditions Through Decomposed Water-Bearing Dolomite.

As stated previously, no evidence of any shear or disintegration was disclosed at the eastern contact between the gneiss and the dolomite; but the western contact, as encountered from the Astoria end, was featured by some 354 ft. (along the tunnel center line) of highly decomposed dolomite with a heavy flow of water. It is interesting to note that, of this 354-ft. belt, the eastern side was advanced to a farther state of decomposition than the western, the rock mass containing countless seams of decomposed material and pockets of residual clay, all so disintegrated and distorted that the cleavage planes were not distinguishable, and though the strike maintained regularity, the dip was most irregular; in fact, at numerous points it was indeterminable.

The seams were so extremely decomposed that the material consisted only of a micaceous greensand, so porous that, on exposure, water quickly formed a free passage. Usually, these seams were little better than loose sand, although at times they choked the rock seam to such an extent as to yield practically no flow of water.

The first contact with the decomposed rock (which was met in driving the Bronx heading) seemed to indicate the likelihood of getting through the contact without serious difficulties, as the tunnel, after careful grouting, was advanced a considerable distance in rock, which, though excessively distorted and water-bearing, had sufficient strength to carry the pressures without danger. At the same time, it was desirable to protect this heading as it advanced, and the arch was carried on heavy timbering in order to guard against falling rock. Having extended this length on timbering, it was thought that the insertion of cast-iron lining for this section might enable the work to advance successfully, particularly as the rock appeared to be getting stronger and harder beyond the first point of contact. As immediate delivery could be obtained of cast-iron segments made from the patterns from which the Pennsylvania North River Tunnel segments were constructed, a short length of 50 lin. ft. was cast, delivered, and promptly

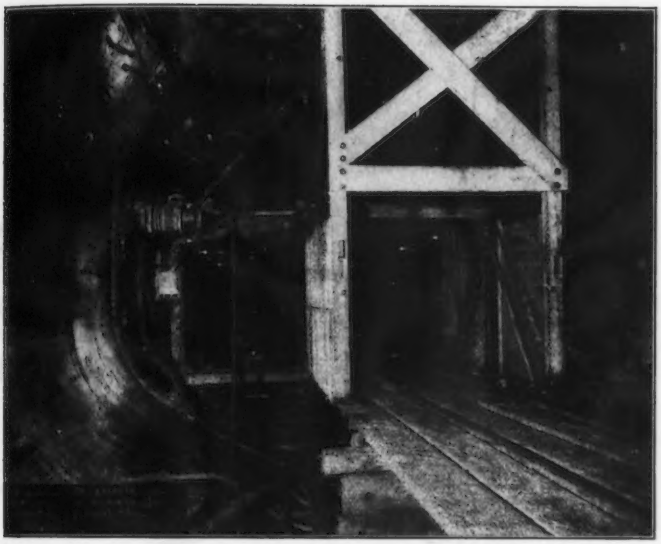
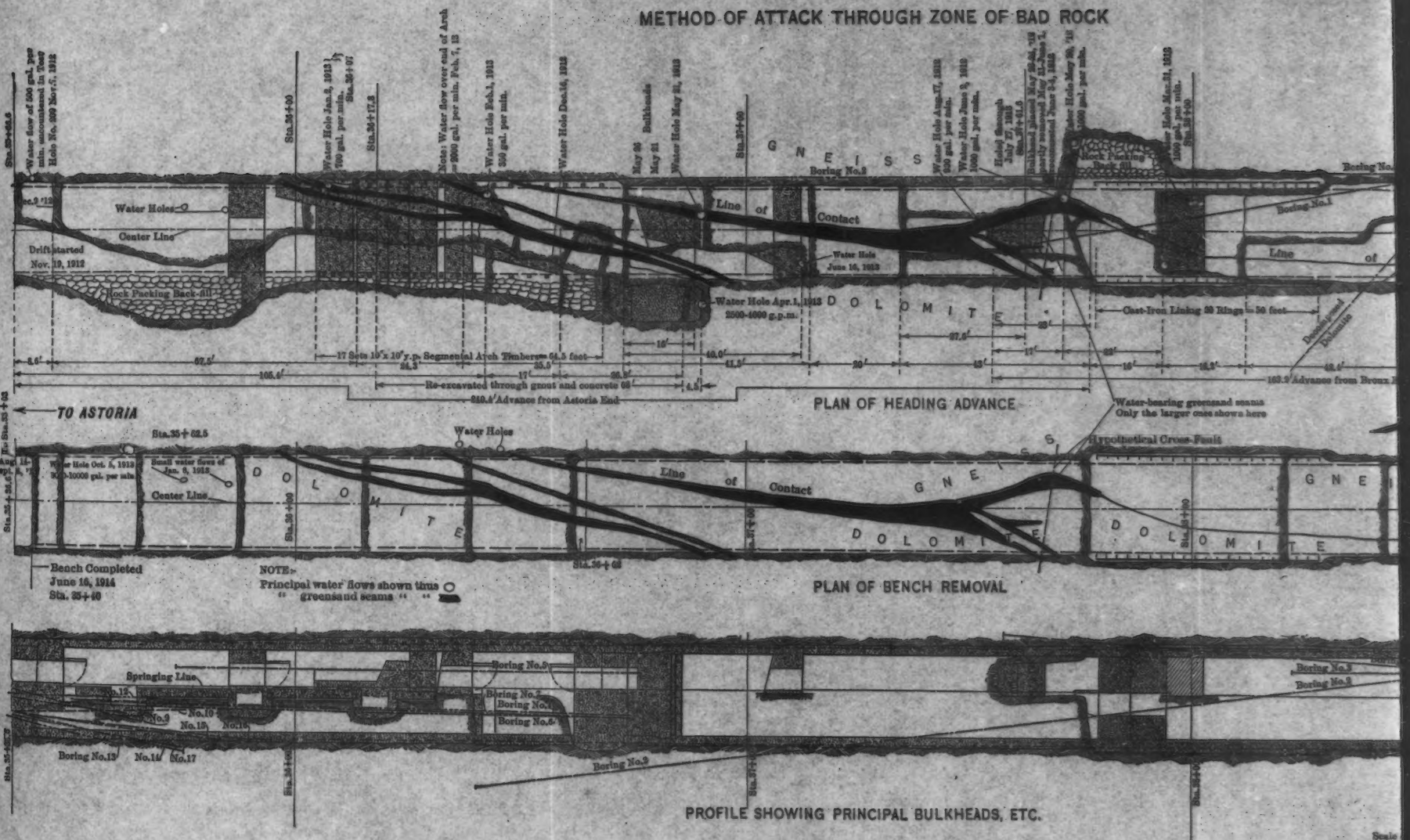
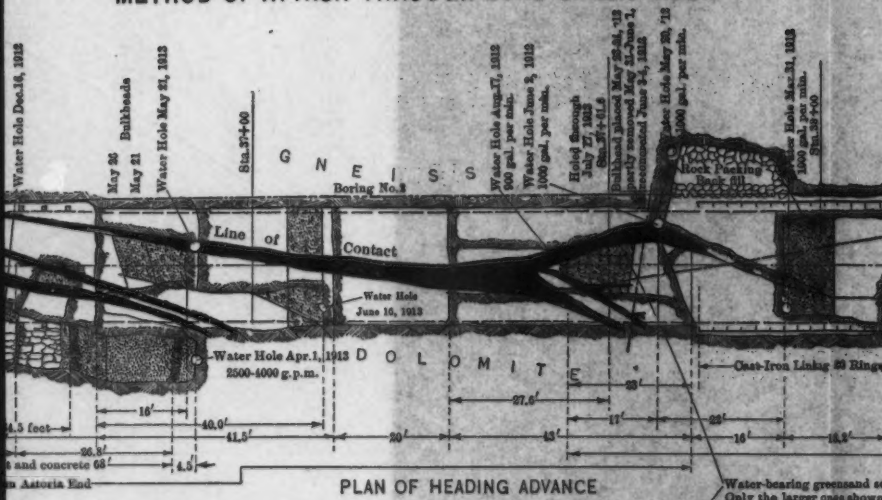


FIG. 9.—STEEL FORM FOR PLACING CONCRETE LINING.

FIG. 10.—HEADING AT STATION 37 + 00, SHOWING SAND HOLE.


METHOD OF ATTACK THROUGH ZONE OF BAD ROCK
Water flow of 500 gal. per min. encountered in Test Hole No. 309 Dec. 7, 1912
Sta. 35+62.5
Sta. 34+00
Water Hole Jan. 5, 1913 700 gal. per min.
Sta. 33+97
Sta. 34+17.3
Note: Water flow over end of Arch = 2000 gal. per min. Feb. 7, 13
Water Hole Feb. 1, 1913 300 gal. per min.
Water Hole Dec. 16, 1912
May 26
May 31
Bulkheads
Water Hole May 31, 1913
Sta. 32+00
G N E I S S
Boring No. 8
Water Hole Aug. 17, 1913 250 gal. per min.
Water Hole June 9, 1913 1000 gal. per min.
Holed through July 27, 1913
Bulkhead placed May 29-31, '13 partly removed May 31-June 2, recovered June 2-6, 1913
Water Hole May 28, '13 1000 gal. per min.
Water Hole Mar. 31, 1913 1500 gal. per min.
Sta. 30+00
Rock Packing Back-fill
Boring No.
Boring No. 1
Line of
Water Holes
Center Line
Drift started Nov. 19, 1912
Rock Packing Back-fill
Line of Contact
Water Hole June 16, 1913
Water Hole Apr. 1, 1913 2500-4000 g.p.m.
D O L O M I T E
Cast-Iron Lining 30 Rings = 50 feet
Dense gray Dolomite
8.6'
97.5'
17 Sets 10"x 10" y.p. Segmental Arch Timbers = 64.5 feet
84.3'
35.5'
38.5'
41.5'
20'
13'
11'
23'
16'
16.9'
48.6'
105.6'
17'
4.5'
Re-excavated through grout and concrete 68'
910.4' Advance from Astoria End
163.2' Advance from Bronx E
PLAN OF HEADING ADVANCE
TO ASTORIA
Water-bearing greensand seams Only the larger ones shown here
Sta. 35+62.5
Water Holes
Hypothetical Cross-Fault
Aug. 11-Sept. 8, '13
Water Hole Oct. 5, 1913 3000-10000 gal. per min.
Small water flows of Jan. 6, 1913
Center Line
D O L O M I T E
Line of Contact
G N E I S S
D O L O M I T E
G N E
D O L O M I T E
Sta. 32+68
Bench Completed June 16, 1914 Sta. 35+60
NOTE:- Principal water flows shown thus
" greensand seams " "
PLAN OF BENCH REMOVAL
Springing Line
Boring No. 5
Boring No. 4
Boring No. 3
Boring No. 6
Boring No. 3
Boring No. 2
No. 12
No. 9
No. 10
No. 15
No. 16
Boring No. 13
No. 14
No. 17
Boring No. 9
PROFILE SHOWING PRINCIPAL BULKHEADS, ETC.
Scale

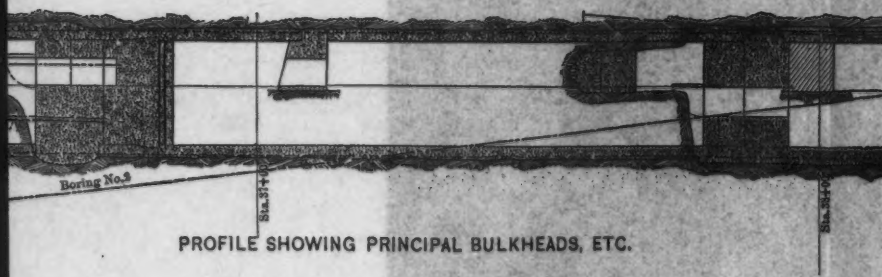
METHOD OF ATTACK THROUGH ZONE OF BAD ROCK



PLAN OF HEADING ADVANCE

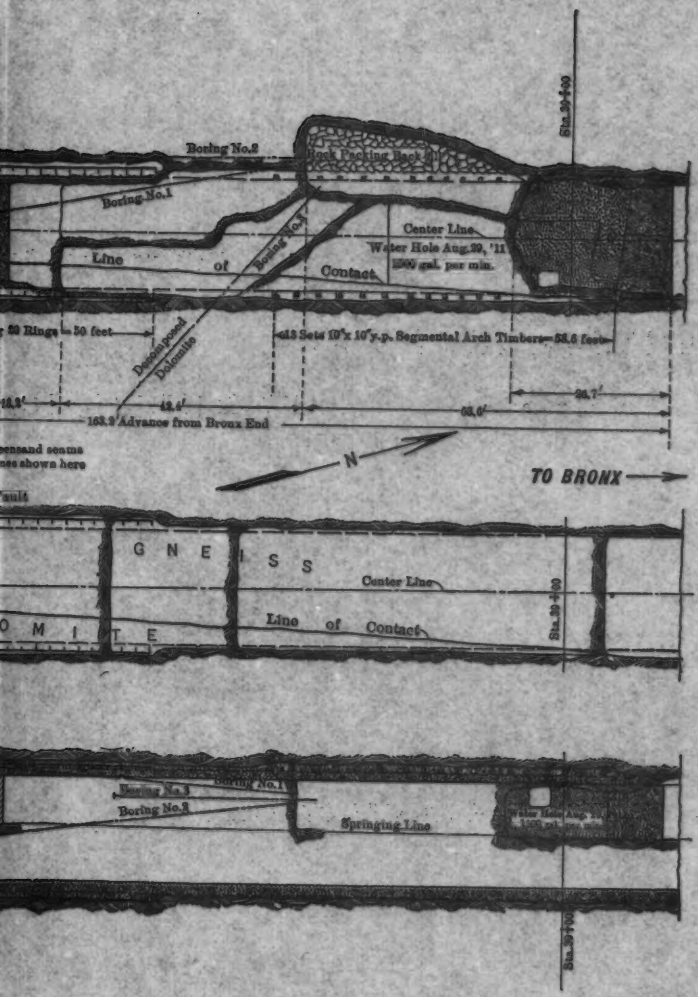
Water-bearing greensand s
Only the larger ones shown

PLAN OF BENCH REMOVAL



PROFILE SHOWING PRINCIPAL BULKHEADS, ETC.

PLATE XXVI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXX, No. 1359.
DAVIES ON
THE ASTORIA TUNNEL.



erected in this wet, water-bearing, rock section. Then, inside the iron lining, concrete was inserted to the neat finished section adopted for the tunnel. This lined section remained in place in the finished work as an intermediate portion between two sections of cast-steel internal lining subsequently erected, which the following record describes.

The seams of greensand developed a maximum width of about 6 ft. at 3 750 ft. from the Astoria Shaft (Fig. 10). In this vicinity also, various pockets in the greensand seams were disclosed; one in particular being 4 ft. square and 20 ft. long, from which the previous water flows had evidently washed the sandy material.

On Plate XXIV, taken from the published Geological Survey, which presents the historical geology surrounding the Astoria Tunnel, it will be observed that there is a non-conformity in the plan defining the positions of the lines of contact as between the north and south sides of the Bronx Kills. This has been accounted for by assuming the existence of a cross-fault between Randall's Island and The Bronx, marking the course of the Bronx Kills. Although no conclusive evidence of the existence of such a cross-fault was observed during the driving of the tunnel, indications of its possible existence and proximity were observed. The material encountered in the vicinity of the assumed cross-fault was so disintegrated and decomposed, and the conditions for observations were so unfavorable, that it is doubtful if such a fault would be readily recognized in passing through it.

It has been assumed, however, that the tunnel passed through this hypothetical cross-fault, as shown on Plate XXVI, this assumption being based on two important facts:

1.—The distortion of the line of contact; and
2.—The rock structure encountered.

(1) It is quite natural that a cross-fault should have a limit of length, so that, in the case of this particular one, though it may extend through the Bronx Kills, as assumed, it may not necessarily extend to the path of the Astoria Tunnel. Such being the case, it seems possible that the actual fault at its eastern end terminates very close to the tunnel path, and that this end, instead of consisting of an abrupt fracture, is a gradual bend or distortion of the rock mass. That such a distortion was passed through is clearly shown on Plate XXVI. This distortion of the actual line of contact between the

hard micaceous gneiss on the west and the badly decomposed dolomite on the east, was encountered by an abrupt bend, 858 ft. from the center of the Bronx Shaft, and extended along the tunnel line some 70 ft. The maximum distortion was 10 ft., at a point midway along its length, at which point the hypothetical cross-fault has been assumed.

(2) The rock structure encountered at this point (893 ft. from the center of the Bronx Shaft), in addition to being extremely porous, was greatly shattered, more so than at any other point in the tunnel. The dolomite, especially, appeared like a mosaic of inlaid tile slabs, quite small (3 to 4 sq. in.), and readily removed by hand, although the individual pieces were moderately hard. It was at this place (Fig. 11) that three large water-flows burst forth at various times. One was an average single flow of 1 000 gal. per min., and in addition to this water, it carried in sediment large quantities of greensand and countless hard rock fragments. Here, too, it was necessary to terminate two heading drifts from the Bronx end. To the north of the assumed cross-fault, the actual contact between the gneiss and dolomite consisted of a continuous small seam, ¼ in. wide, of soft brown mud, and to the south of this point the contact plane was featured by a 3 to 4-ft. seam of hard, firm, bluish-green clay.

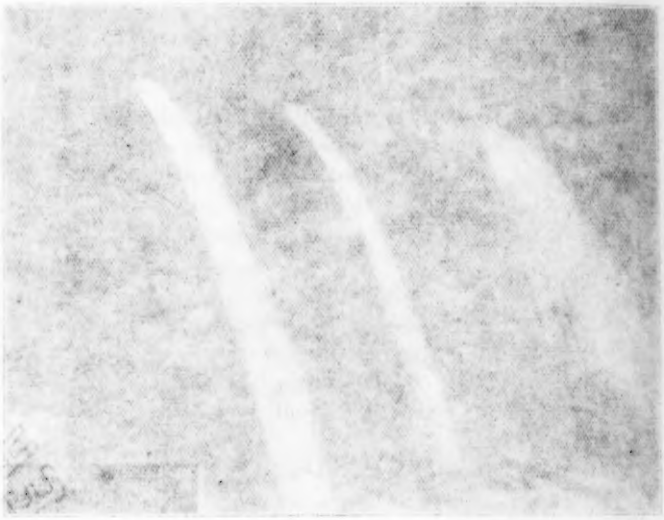
In this section of New York City, observation has established the existence of cross-faults, one at 138th Street and St. Nicholas Avenue, and one parallel to it, south of Hell Gate, and crossing the East River north of Blackwell's Island.

Water Flows.—The water-bearing fissures in the 354 ft. of decomposed dolomite were quite numerous, sixteen appreciable streams, each averaging from 300 to 10 000 gal. per min., under a full pressure head of 95 lb., having been encountered. These flows were usually quite sudden, and were accompanied by an inrush of greensand, the maximum flow of 10 000 gal. alone washing in 400 cu. yd. of sand, coal, etc., and, during a period of 6 weeks, some 1 700 cu. yd. of sand were washed into the Astoria heading. The establishment of a direct connection to the river bed was proved by the appearance of air bubbles on the river surface during grouting operations in the headings, as well as by the quantities of coal, wood, shells, bricks, etc., washed in at various times.

Method of Attack.—On account of the great depth of the tunnel and the resultant pressure head (95 lb. per sq. in.), any compressed-air methods of attack were out of the question, and, on account of the

FIG. 11.—WATER FLOW IN DRIFT AT STATION 37 + 71.

FIG. 12.—GROUT PIPES, STATION 38 + 55 TO STATION 39 + 00.



necessities of the gas supply, it was not desired to go to any lower level in an attempt to pass under the disintegrated bed of dolomite. It was consequently necessary to drive the tunnel at the original depth and alignment, so that the proposition presented was a difficult one.

The theory on which tunneling operations were executed through the decomposed rock formation was the thorough exploration of the formations ahead by test holes and the consolidation of the rock by grouting. It was very obvious, in working in a narrow top heading, even by drilling numerous holes in the form of a fan, that a very limited area could be reached for consolidation, and as the percussion drill, working on more or less horizontal or dry holes, is only capable of extending to a depth of about 20 ft., the tunnel section which it is possible to consolidate is very short, particularly in view of the fact that there is a definite limit to the number of grout holes which can be drilled into the face of such a small heading (Fig. 12), and that considerable time must be allowed for cement grout to attain strength to permit of blasting safely. It was obvious, therefore, that a greater area of attack must be provided, and this was done on the theory of diverting the heading from the straight line to lines external to the final side lines of the tunnel, following the line of the strike of the sound rock as far as possible to permit of cross access to the decomposed strata. By doing this a long face could be exposed from which the drilling could proceed. Further, whenever the rock was sound and reasonably hard, a full and adequate anchorage for the grout pipes was naturally provided, but, if the rock was decomposed, seamy, and water-bearing, it was usually found to be so soft that proper anchorage could not be obtained for the grout pipes. In such cases it became necessary to construct buttress faces to the soft rock after exposure, before grout pipe holes could be drilled and pipes inserted. The work was carried out on the further theory that grouting above the extrados of the arch and outside the side-walls, within the limit of the top heading, would in all reasonable probability cut off the direct access of heavy water flow and consequent danger of breaks occurring in the bottom. This was carried out to the end of the work and proved successful, as was illustrated by the fact that it was found unnecessary to do any grouting of considerable extent in order to take out the later bench excavation, the water-bearing seams having evidently been

plugged at the higher level and the resistance of the decomposed material in the seams being adequate to resist any indirect access or flow.

Reference has been made to the necessity for buttressing the face of soft rock to provide anchorage for grout pipes, in the event of the rock being unsound and not giving adequate security for the insertion of such pipes. In such cases, owing to the high pressures involved, these concrete buttresses were allowed to set for practically two weeks on each occasion before they could be utilized.

When such buttresses were set and thoroughly hard, or when sound rock existed, the process of preparing the grout pipes for use, as indicated by Fig. 13, was generally as follows:

The drilling was commenced with 6-in. percussion drills, and entered from 3 to 4 ft. deep into the concrete buttress or hard rock. A piece

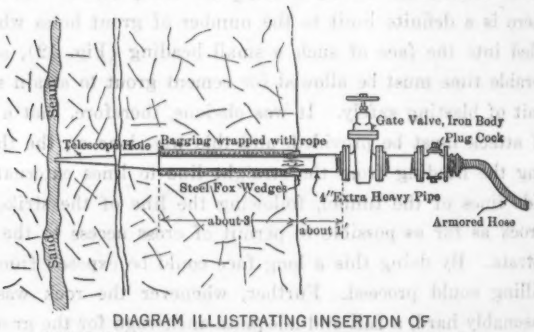


DIAGRAM ILLUSTRATING INSERTION OF GROUT PIPES IN ROCK FOR HIGH-PRESSURE GROUTING.

FIG. 13.

of 4-in. pipe, about 1 ft. longer that the depth of the hole, was then wrapped tightly with bagging and wound around with marline to secure it tightly to the pipe, the quantity of bagging being put on up to practically the dimensions of the 6-in. hole. The wrapped pipe was then inserted in the hole, and driven to its extreme end, no thread, flange, or other protector being put on the inner end of this first length of pipe. When thus inserted the wrapping was caulked back into the hole, and steel wedges were inserted and driven tightly between the 6-in. hole in the rock and the body of the 4-in. pipe. These wedges were driven so hard as to indent the 4-in. pipe and secure it rigidly. A screwed flange was then put on the outer end of this 4-in. nipple and an open gate-valve was attached to the flange, in order to provide

a clear opening equal to the diameter of the pipe. Drilling then proceeded through the gate-valve, commencing with 3-in. percussion drills and telescoping if necessary, as the drilling advanced. The length of the test holes rarely exceeded 20 ft. Of course, it will be understood that where the drilling was carried out from narrow cross-headings, and the holes were drilled at an angle with the strike, the effective length was reduced on account of the lack of clearance to insert long drills.

Reference is made in the following pages to the use of the diamond drill. On first meeting the decomposed rock at the Bronx end, a 1⅛-in. diamond drill was set up in the heading and holes were driven, one at the elevation of the heading, horizontally at right angles to the strike of the rock, in order to intercept the seams at right angles to the general strike; and two holes straight ahead on the axis of the tunnel, one horizontally and one pointing downward. The hole at right angles to the strike of the rock was extended in only some 50 ft., but the long holes ahead were extended for about 200 ft., and these drill holes gave an excellent idea regarding the conditions to be met, as far as drilling could be accomplished with the small machine used. At a later date, a heavier diamond drill, capable of drilling 3-in. holes, was used, and gave much more satisfactory results. The use of the diamond drill for long holes was extended considerably, previous to and after the flooding of the tunnel, giving long penetration and providing a means for grouting at considerable distances. The later diamond drill outfit was of the Sullivan type, of heavy construction, and proved to be admirably adapted to work of this character. In the entire construction through this decomposed rock section, some 3 300 test holes were drilled, aggregating a length of about 43 300 ft. by percussion drills, in addition to seventeen diamond drill borings aggregating a length of 1 500 ft.

Grouting in the Astoria Tunnel has probably exceeded very greatly that in any other work executed, in consolidating soft fissured rock, and in stopping extensive and large leaks at high pressure. Grouting at a higher pressure was done on the Catskill Aqueduct, though the volume of inflow was probably very much less. In the case of the Astoria Tunnel, the grouting was a continuous operation for consolidating and cementing decomposed rock throughout the entire length of the sheared contact between the gneiss and dolomite, and as such

served the purpose, a result which has not, it is thought, been approached or attempted in other work. The work involved was the conversion of a rotted and decomposed rock filled with fissures into a solid and substantial substance through which the tunnel could be driven and, incidentally, at the same time, stop the influx of water. In the grouting operations it was found to be almost impossible to grout into a seam of decomposed dolomite sand. This would apply equally to any other sand or material filling seams, which is likely to be scoured out by a flow of water. On meeting a seam filled with decomposed rock sand, the indications on this work were, that the sand was practically impervious, but, after a short exposure to water flow, the soft rock abutting on the seam became water-loaded and softened; then the passage of small quantities of water loosened and demoralized the sand filling the seams, following which the sand was scoured out, thus forming an open water channel. Therefore, it was found particularly desirable, after drilling grout holes into a seam filled with sand, to rake the sand out with rods, as far as was feasible, and then to allow these test holes to flow freely, by blowing with high-pressure air or in any other way, the idea being to get the sand to flow with the water and thus empty the seam, after which the grouting operation was much more thorough, successful, and permanent. When these seams were emptied of the sand and grouted at high pressure the result was practically a solid rock formation, and the tunnel could be built by blasting operations with security; whereas a seam not so clear of sand would resist the flow of grout and be a menace to the future advance of the tunneling operations.

Glass Tunnel Model.—In order to present clearly the actual geological information, disclosed both by direct observations and test-hole exploration, so as to determine the most advantageous direction of drift attack, an unusual method was designed by the field engineers and applied to this work. This consisted of a glass plate tunnel model. As the general strike of the rock containing the numerous seams of disintegration was nearly parallel to the tunnel line, and the dip was decidedly irregular, it was impracticable to plot the geological data on drawings and still have them clear and of working value, as they could not conveniently be superimposed. This condition was admirably simplified by this glass plate model, constructed at the tunnel works. This method of geological plotting proved quite successful, as

the conditions of the explored rock structure were clearly presented at a glance, proving an important factor in the determination of the various methods of attack adopted.

This model consisted of a series of glass plates, 16 in. square, placed upright in a skeleton frame, 12 ft. long, and arranged to scale corresponding to the 5-ft. tunnel stationing. As the tunnel driving proceeded, the actual lines, drifts, bulkheads, timbers, iron rings, etc., together with the exposed disintegrated seams and the test-hole information, were all painted with oil colors on the plates, this work being always maintained up to date, so that the conditions actually existing were always visible.

Blasting was naturally performed with very light charges, necessitated by the unsound strata, as water flows were easily started, not only by the shattering of the rock in their immediate vicinity, but by the concussions from blasting in sound rock at a considerable distance from the dangerous strata. In one instance, at the Astoria end, a 50-ft. length of drift was temporarily lost by this latter condition. The caution and care with which the rock was thoroughly sounded and scaled immediately following blasting is shown by the fact that no serious injuries or fatalities occurred, which is regarded as unusual, in view of the extremely unsound condition of the rock.

Grouting.—During the early stages of adverse tunneling, the grouting of the water-bearing fissures was performed with ordinary grout machines, using a neat Portland cement mortar at a pressure varying from 100 to 200 lb. per sq. in. This made it necessary, in September, 1911, to put in a steam-driven, straight-line booster at each plant. Grouting continued with this pressure for some months, but it became apparent as the work proceeded that an air pressure of even 200 lb. was insufficient to insure success.

For successful grouting into narrow seams it is essential to obtain a high initial velocity of grout flow, in order to overcome the frictional resistance; consequently, to grout against a water head of approximately 100 lb. per sq. in., more than 100 lb. additional pressure was found to be necessary. The increase of the starting pressure insured the initial flow, and the expansion of the air, even down to 200 lb. or less, was adequate to maintain the continuity of the flow. The use of the high pressure also insured that the seam would be packed

more tightly and that the grout would be more dense when it had taken its ultimate set and attained strength.

Consequently, an additional steam-driven straight-line booster was erected at the Astoria end in November, 1912, this machine being capable of developing an air pressure of 500 lb. per sq. in., for which special grout machines were constructed. The injection of cement into the water-bearing fissures at this pressure (500 lb.) was exceptionally effective, and was undoubtedly a valuable factor in the ultimate success in driving through the unsound water-bearing rock.

The disintegrated rock structure (Fig. 14) frequently necessitated the construction of concrete wall buttresses, in order to withstand the high grouting pressures used, and considerable difficulty was experienced during the early stages of the work in preventing the injected cement from washing out of the water-bearing crevices. Unsuccessful efforts were made to prevent this by mixing oats and bran with the cement mortar, as used in other subaqueous tunnels.

Another method introduced, which proved of great benefit and aided in securing the injected cement, consisted of the addition of a handful of fine-cut cotton waste to each batch of grout, which by fibrous reinforcement of the cement held it in place while it was setting. During lengthy grouting periods, two machines were used alternately in series, thereby maintaining a continuous injected flow.

The special machines for grouting with 500 lb. air pressure were designed and assembled at the tunnel works. The general arrangement and capacity were identical with the riveted pans for 200 lb. pressure, with the exception of some details designed to resist the higher pressure. The entire pan and the cover were made of cast steel, carefully annealed and tested, and having no joints other than that of the cover. The heads were made in convex form; one end was cast in one piece with the body, and the other was a cast-steel removable cover with male and female bolted flanged joints. In operation these pans were very reliable, and, when required, one pan was operated 25 hours without a stop, and discharged 4 500 bags of cement at pressures varying from 350 to 500 lb. Extra heavy 2-in. screwed pipe was used for the transmission of the high-pressure air from the engine-room to the heading, with the exception of the first 600 ft., where extra heavy 4-in. pipe was used to give greater storage capacity. Special 1-in. hose was used to make the connection between the air line

FIG. 14.—FACE OF HEADING AT STATION 36 + 72.5.

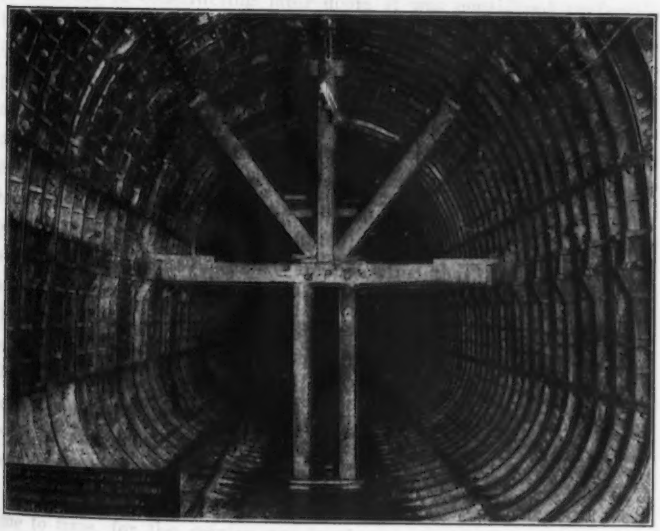
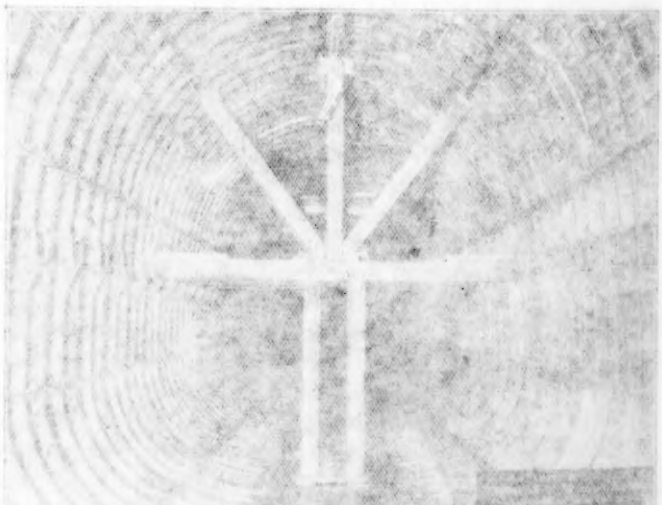


FIG. 15.—TIMBER BRACING BETWEEN BULKHEADS.



and the grout pan. This was a 6-ply rubber and canvas hose, with a marline woven cover, wound with $\frac{3}{16}$-in., half-round, steel wire, $\frac{4}{10}$-in. pitch. Knox standard couplings were fitted to each end. A special 2-in. grout hose was used, this also being of 6-ply rubber and canvas, wound with $\frac{3}{16}$-in., half-round wire, $\frac{1}{2}$-in. pitch.

Bulkheads.—Prior to entering the belt of disintegration at each end, adequate precaution was taken to guard against flooding, by the construction of full tunnel section emergency bulkheads, well back from the point of attack. As the headings proceeded, additional emergency bulkheads were built as close to the heading as the work permitted. These were to the heading size only, as the bench was not removed through this soft-ground section until after the meeting of the headings.

The first emergency bulkhead of full tunnel section, built at the Bronx end, was of a concrete buttress type, fitted with two guillotine doors and not reinforced. A guillotine type of door was considered in the first design to be the most certain in action. The clear openings of these doors were about 5 ft. square, of such size as to permit a tunnel car to pass through. These doors were never actually closed or used, but in constructing later doors it was considered preferable to adopt a simpler form of top-hinged flap, so that, in case of necessity, by cutting away the wire rope supports, they would drop readily into position, this type being not only cheaper to build but simpler in operation, and throughout the remainder of the work such doors were used. Some of the full tunnel section bulkheads, as well as the emergency bulkheads, were reinforced and some were not, the design being modified for each individual case. These doors were used only occasionally, and, as hereinafter stated, gave excellent results in operation. Owing to the necessity for knowing that these doors were always in working order, and for trying them out from time to time, they were equipped on the outer side with tripping devices and rams operated by compressed air from the tunnel line for raising them when they were desired to be opened. Valve arrangements were provided so that the door could be operated from inside or outside; thus, in the event of any one being accidentally left on the wrong side, it was possible for him to open the door and release himself.

Numerous solid heading and wall buttresses were constructed from time to time, for the reasons previously stated. This concreting, of

course, necessitated frequent and unusual delays, the accumulated lost time incurred by this work accounting for at least 50% of the time required to tunnel the 354 ft. of unsound rock, as generally from 10 to 14 days were allowed for the concrete to set sufficiently for 500 lb. grouting pressure.

During the bench removal, after holing through the headings, full section tunnel emergency bulkheads were necessary, these being generally about 14 ft. thick. As the bench excavation proceeded, these bulkheads were kept fairly close to the point of work, in order to facilitate transportation. The continual construction of heavy timber runways was necessitated by the fact that it was desirable to place the emergency bulkhead doors well above the invert, in order to prevent them from becoming clogged by the sandy sediment in the inflowing water. Drainage through the bulkheads was provided by 6 and 8-in. pipes, passing below the doorways, but at a sufficient height from the bottom to prevent them from being clogged by the sand. These drains were equipped on the shaft side with standard gate-valves to permit of their being closed.

Bench Excavation.—After the meeting of the headings, on July 17th, 1913, efforts were concentrated on the removal of the remaining bench, which, at that time, amounted to 474 lin. ft. This length of excavation was attacked simultaneously from the Astoria and Bronx sides. In the first 237 ft. on the Astoria side only good sound dolomite was encountered, no trouble was experienced, and the immediate lining of this excavation was not necessary. On the Bronx side the lower west portion of the bench face consisted of hard sound gneiss with intrusions of clay at its contact with the dolomite on the east. The dolomite was both porous and disintegrated, but no serious difficulties were encountered, although it was essential to place the concrete lining immediately after each short advance of excavation.

In this bench excavation the advance was thoroughly explored with long test holes ranging well outside the necessary excavation lines, and the material was displaced by light blasting. This arrangement was due, not only to the character of the rock, but on account of the existence of the overlying concrete arch.

The work proceeded in this manner until exploratory test holes disclosed the presence of excessive water at the Astoria side of the remaining bench. This water was located by four test holes, 3 ft.

apart, along the west side of the bench top. The nearest hole was 24 ft. from the bench face, and the total flow at high pressure was about 1 500 gal. per min. These holes were piped for grouting, and encased in an inclined concrete buttress, 20 ft. long and extending from the arch haunch to the rock bench.

The water was allowed to flow freely through these holes for one week, while the concrete buttress was attaining strength. The valves of the drains through the inclined buttress were then closed in preparation for grouting, and water broke through the top of the bench at the toe of the buttress. It was then decided to place a full-section emergency bulkhead at each end of this remaining 122-ft. section of bench, so that any water that might overcome working operations could be confined between the bulkheads.

On the completion of these emergency bulkheads, on September 16th, the work of preparing to consolidate the rock previous to excavation was resumed. The inclined buttress was removed and a much heavier and more massive one was constructed. This extended along 34 ft. of the bench top from the recently constructed emergency bulkhead on the Astoria side, was about 6 ft. high and 12 ft. thick, and was completed on September 25th.

At the same time, the western side of the bench top, at the Bronx side, was covered with a 23-ft. length of inclined buttress against the concrete arch and extending from the recently constructed Bronx emergency bulkhead to a heading emergency bulkhead on top of the bench placed during the driving of the Astoria heading. A similar inclined buttress, 13 ft. long, was placed against the west side of the arch. Thus, the entire 122 ft. of exposed bench was covered on the west side by a series of concrete bulkheads and buttresses.

After allowing these structures to set until October 1st, grouting through them into the rock fissures began, and 139 bags of cement were successfully injected into the last two mentioned buttresses. The grouting of the massive 34-ft. buttress was not successful, as the reactive pressure developed by the closing of the drain valves fractured this concrete structure very badly and rendered it useless for the purpose for which it was intended.

It was then decided to remove this fractured buttress entirely and replace it with one in the form of an inverted arch, and of more massive proportions, the inverted arch to react against the permanent roof

arch already in place. In the removal of this buttress blasting was done with extremely light charges, and every care was taken not to disturb the underlying rock bench. The removal of this buttress was completed about 8 A. M., on Sunday, October 5th, 1913.

7.—Flooding of Tunnel.

Shortly after 8 A. M. a large flow of water, carrying greensand, burst forth under high pressure from the top of the rock bench about 15 ft. from the Astoria emergency bulkhead. After flowing 30 min. the flow abruptly ceased, only to burst forth again a few minutes later with renewed vigor, yielding a volume of approximately 10 000 gal. per min., which eventually overcame the capacity of the pumps at the Astoria end. The bulkhead doors were promptly closed. On the Bronx side the force of the water was broken by the intervening heading bulkhead still in place, and the workmen were able to close the drains; but, on the Astoria side, owing to the rush of water and sand, they were able to close only two of the three drain valves in the bulkhead. It was found later that a small piece of 1-in. board, floating in the water, had become wedged between the door and the door frame of the bulkhead on the Astoria side. Through the small opening thus caused, and through the drain pipe which the men had been unable to close, the water poured into the tunnel, carrying with it a large volume of dolomitic sand and débris. The water in the Astoria Shaft rose steadily until, at 3.15 P. M., it had completely submerged the pumps, which soon thereafter ceased working, and the water reached tidal elevation at 8.15 P. M., on Monday, October 6th. At this time the pumping plant at the Astoria Shaft consisted of 12 pumps with a rated capacity of 7 500 gal. per min., but, on account of the sandy water continually handled, it had previously been found that the actual discharges were about 70% of the rated capacity; thus the actual capacity probably did not exceed 5 500 gal. per min.

Immediately following the flooding of the Astoria end, the Bronx emergency bulkhead was strengthened as a further measure of safety by a reinforced central buttress. On the completion of this buttress, an additional emergency bulkhead was built about 105 ft. nearer the Bronx Shaft. The actual need of this bulkhead never arose, as the first one withstood the full hydrostatic pressure, but it was desired to take every possible precaution against the flooding of the remaining

protected tunnel, as the unwatering of an entirely flooded tunnel would have been a much more difficult and costly proposition.

It was also considered advisable to increase immediately the Bronx pumping plant, which at this time consisted of five pumps with a rated capacity of 2 250 gal. per min., of which only 1 200 gal. capacity was available for shaft discharge to the surface, the remainder handling the drainage from the bench face to the shaft, as the grade at the Bronx end was away from the shaft. Four additional pumps, each having a capacity of 600 gal. per min., were put in, by which the capacity of the tunnel drainage to the shaft was increased to 1 650 gal. per min.; and the surface discharge capacity was raised to 3 000 gal. per min. This plant was fully capable of handling all the subsequent water from the Bronx end.

8.—Recovery of Tunnel.

Observations of the rise of water in the Astoria Shaft were carefully made, and also the tidal readings during the continuance of the shaft inundation. It was observed that the water rose and fell with the tide, but that the entire tidal rise was not obtained, the rise in the shaft being only about one-half that in the river. From the continuance of these conditions, and also from the flow obtained through small pipes in the Bronx bulkhead, it was apparent that the water flow had not choked itself as had been hoped. After due consideration of various methods of attack, it was decided to grout from the Bronx end into the water-filled chamber between the Astoria and Bronx bulkheads, with the object of filling this with cement to the extent of permitting the unwatering of the tunnel from the Astoria end. At the same time diamond-drill borings were to be driven from the Bronx bulkhead, the purpose of which was to intercept the fissure at a point beyond its opening into the tunnel, carefully calculated and directed to the desired point, and through this to inject cement, in an effort to seal the source of flow beyond the tunnel. The first hole drilled was intended to tap the fissure at a point 5 ft. below and 5 ft. beyond the point of flow into the tunnel, this location being determined by a close study of the geological formation previously encountered. This hole measured 120 ft. from the bulkhead face to the point. The accuracy of both the location and pointing of this hole was conclusively proved, not only by the injection of 870 bags of cement, but by the conditions

disclosed when driving through this section in February, 1914. The grout in this instance was evidently well directed into the fissure, but naturally followed the line of least resistance, going no farther than the débris in the fissure would permit.

The second step in the programme of attack was to fill completely the space between the completed arch and the rock bench between the bulkheads, and began by drilling through the Bronx bulkhead and the small disused heading bulkhead with a diamond drill, using a packing tube fitted with an exterior valve inserted in the bulkhead, through which the diamond drill rods were threaded and operated. These drill rods were pushed into the chamber as near as possible to the fissure location. Grouting through this diamond-drill tube began at 3 P. M., on November 10th, and continued without interruption until 4 P. M., on November 12th, during which time 8 580 bags of cement were injected at a pressure of 400 lb. per sq. in. The drill head, fitted with its diamonds, was recovered later from the mass of cement grout during the subsequent excavation. The grouting was discontinued on the appearance of cement in the seepage around the Bronx bulkhead door.

From the action of the water in the Astoria Shaft, it was quite evident that this grouting had been effective in sealing, to a large degree, the direct passage and subsequent leak between the tunnel and the river, but had not entirely sealed the leak through the Astoria bulkhead, as the draining of water through numerous small pipes in the Bronx bulkhead lowered the water in the Astoria Shaft some 17 ft. This water receding in the Astoria Shaft, and its subsequent loss of pressure head, resulted in an unequal condition of pressures on the freshly injected grout in the fissure, so that, when an absolute pressure of 11 lb. existed on the fresh grout, from the drop in the Astoria shaft water, the grout gave way, causing the water from the fissure to flow again into the tunnel, and the shaft water to rise.

Meanwhile, the long tubing was kept free, both inside and outside, and was withdrawn for this purpose to within 7 ft. of the inner bulkhead wall. It was again pushed forward in the early morning of November 14th, and 775 bags of cement were injected. Later the same morning, 622 bags were injected, followed in the afternoon by 986 bags. On the following morning 316 bags were injected, when the pressure gauge on a pipe extending through the bulkhead began to

jump in excess of the hydrostatic pressure, and, as grout began to appear through the bulkhead pipes, the operations were discontinued. Again, however, on November 17th, an additional 164 bags were injected through this tube, making a total of 11 342 bags of cement.

Meanwhile, the diamond drill borings had been continued, and, although water and greensand were encountered, only a small quantity of cement could be injected. It was evident, later, on removing this piece of bench, that the fissure in question was pierced only by the first drill hole. By opening the various pipes in the Bronx bulkhead, in order to test the hydrostatic pressure and to ascertain the effect of the grouting, it was found that they gave water, but without pressure, and, therefore, it was considered quite safe to commence immediately the unwatering of the tunnel from the Astoria end.

The consideration of the various possible alternative methods for unwatering the tunnel contemplated in every case the ability to hold the Bronx tunnel intact up to the emergency bulkhead, and the method adopted and subsequently carried out was based on that understanding. However, it was also considered as a possibility that this method might not prove successful in stopping the open leak from the river so as to enable a battery of pumps, which could be practically erected, to control the flow. As the heading had been entirely carried through and the test borings had defined quite clearly the position of the seams, it was thought that, if the plan of drilling with diamond drills, as previously described, and grouting through the diamond drill holes, did not prove successful, another alternative would still remain, by which a steel trestle could be erected over the tunnel on the bed of the river and from the deck of such a trestle, above high-tide level, numerous drill holes could be accurately placed and extended vertically to intercept the line of the seam, and through such drill holes grouting could be executed at any pressure desired, which would be amply adequate to fill the seam and plug it effectively. Fortunately, this last expedient was not found necessary.

Unwatering.—Existing in the Astoria shaft prior to the flooding were an 18-in. ventilating pipe (extending from the surface, down the shaft, and into the tunnel a distance of 970 ft.), and three 8-in. discharge lines (extending from the pumps at the foot of the shaft to the surface). These four pipes were converted into a pneumatic water-lift system by placing a 3-in. air pipe inside the 18-in. ventilating pipe,

extending the former to a depth of 251 ft. below the ground surface, or a depth of 240 ft. below mean sea level, and by placing a 2-in. air pipe inside each of the 8-in. lines to the same depth. The discharge ends of these air lines, for some 24 in. in length, were perforated with small holes drilled at an angle of 45°, so that the air discharge would point upward. These air lifts all discharged at the shaft surface. This emergency equipment was the only means used to lower the water 176 ft. from the top of the shaft, and down to that depth, proved both rapid and effective.

During the operations at the Bronx end, an emergency plant had been secured and prepared for operation at the Astoria Shaft. This consisted of six Cameron reciprocating pumps (one of 1 000, one of 800, and four of 600 gal. per min., a combined rated capacity of 4 200 gal. per min.). On a steel pontoon, designed and constructed specially for the purpose, and floating in the shaft, were placed the two emergency pumps (of 1 000 and 800 gal. per min.) with the discharge connected to an 8-in. discharge line in the shaft. Three 600-gal. pumps were placed in the elevator wells after the water had been lowered 172 ft. below mean sea level. These pumps were lowered from time to time as the water dropped, and, when it had been lowered to a level of 13½ ft. above the tunnel invert at the shaft, the submerged permanent pumps were brought into play.

The 18-in. and two of the 8-in. air lifts were operated continuously from 8 A. M. to 9 P. M., on November 24th, when the water had been lowered a depth of 160 ft., displacing 1 147 028 gal. of water in 797 min., or an average discharge of 1 439 gal. per min. The two pumps on the pontoon were then started and worked continuously until 6 A. M., on November 25th, with a net gain of 15 ft. The three pumps in the cage wells were then operated, it being necessary to change the pipe-line connections quite frequently. On November 30th, at 10.40 A. M., the water had been lowered to a level 13½ ft. above the tunnel invert at the shaft portal. During the progress of this stage of the pumping, in order to avoid a column of water forming at the Astoria bulkhead and a consequent probable regurgitation of the water in the shaft, air under pressure was forced into the tunnel through a 2-in. pipe, which was in place and extended from the surface about 3 200 ft. into the tunnel. At 10.40 A. M., on November 30th, the original shaft pumps were started, and, as this gave in all a battery of eleven pumps, the

water was quickly drained from the tunnel. During the period occupied in pumping at the Astoria end, a 2-in. and a 5-in. pipe in the Bronx bulkhead were kept flowing, and materially aided in the unwatering. Table 6 is a summary of the water pumped and the time occupied.

TABLE 6.—SUMMARY OF WATER PUMPED, ETC.

AIR LIFTS—ASTORIA END:	
Time worked	13 hours 17 min.
Volume of shaft unwatered	1 147 028 gal.
Average per minute	1 439 gal.
PUMPS—ASTORIA END:	
Time worked	162 hours 53 min.
Volume of shaft and tunnel unwatered	8 415 353 gal.
Leakage into tunnel	15 608 873 gal.
Total water pumped	24 024 231 gal.
Average per minute	2 458 gal.
AVERAGE LEAKAGE INTO TUNNEL THROUGH THE FISSURE DURING THE PERIOD OF UNWATERING:	
Flow through pipes, Bronx bulkhead	938 gal. per min.
Leakage through Astoria bulkhead	997 " " "
Average leakage through fissure	1 935 " " "
Period of unwatering approximately 25 000 000 gal.	7½ days.

On Sunday evening, November 30th, with the water 4½ ft. below the crown of the arch at the portal, two boats containing the tunnel engineers entered the tunnel and continued on a tour of inspection to the Astoria bulkhead. It was found that comparatively little damage had been done by the flooding, although a considerable quantity of grout and sand had entered the tunnel through the door which, in the inception of the flood, was not securely closed, and through the 8-in. drain which the men had been unable to close, as previously mentioned. Sand deposits occurred at frequent intervals throughout the first 2 300 ft., and beyond this a layer of sand, mud, coal, shells, etc., covered the tunnel invert, accumulating in quantity toward the bulkhead. At a distance of 3 000 ft. from the shaft, the grout, with a small proportion of sand, formed a bank 12 in. deep, gradually sloping up to the bulkhead, a length of about 500 ft., where it was 12 ft. thick, or nearly to the top of the bulkhead doorway. The surface of this grout bank was in a plastic condition, some 6 in. deep, the underlying material being firm and semi-hard, the hardness increasing toward the bulkhead. This material was not well set-up grout by any means, the subsequent removal being performed with pick and shovel. From the 8-in. drain clear water, at the rate of 900 gal. per min., was flowing, and from a 3-in. cable conduit pipe in the bulkhead there was a stream

of 100 gal. per min.—neither being under pressure. The air throughout the tunnel was exceptionally good.

The construction of an emergency bulkhead about 60 ft. from the closed bulkhead was immediately commenced, before making any attempt to remove the aforesaid grout bank, as the appearance and condition of the grout bank material were such as to cause very grave doubt that the leak would remain confined. A small trench was excavated in the grout bank around the bulkhead drains; the two closed ones were blind-flanged and the open one was allowed to flow, in order to prevent the development of any additional pressure on the bulkhead.

Simultaneously with the construction of the emergency bulkhead, the cleaning and righting of the tunnel proceeded. Meanwhile, the emergency pumps were erected at the shaft bottom and the original pumps were overhauled. When this work was completed the Astoria plant consisted of fifteen pumps, with a combined rated capacity of 9 600 gal. per min., which was considered quite sufficient to handle any likely flow, in view of the existence of an additional bulkhead removed from the point of danger.

The method of procedure adopted for completely sealing the water flow of October 5th consisted in driving a series of long test holes, in an effort to pierce the fissure well outside of the tunnel lines, through which the effective seal and solidification of the rock mass would be obtained by injecting grout. The short holes were drilled with percussion drills and the long ones with diamond drill equipment through the tunnel side-walls close to the bulkhead, as it was desired not to disturb the latter in any way. The direction of these holes was based on the assumption that the water-bearing fissure would be encountered in a plane produced along the line of the general strike of the rock and passing through the ends of the holes by which the presence of the fissure was first determined prior to the flood. The first hole drilled with a percussion drill to the computed depth, encountered water at full pressure at a depth of 22 ft.; the success of this hole was very encouraging, as it had been computed that water would be encountered at 21½ ft.

The observation of results attained in grouting was accomplished by the attachment of pressure gauges on both the Astoria and Bronx bulkheads and by allowing the pipes in the Bronx end to remain flowing. Grouting began at 1 P. M. on December 18th, and was followed

immediately by the appearance of grout in the flowing Bronx pipes, thus indicating that the point of injection communicated to the water hole of October 5th, and that the latter was still unsealed. At 9 P. M. on December 19th, after the injection of 3 360 bags of cement, the hole finally refused.

It was quite apparent that the recent grouting had been effective, but, during the time interval necessary to allow this to set thoroughly, it was decided to make a thorough investigation of the underlying rock before attempting to open up the grout-filled chamber between the bulkheads. This investigation proceeded during the remainder of December, and consisted of drilling ten long diamond drill holes, the result being that, though the rock structure outside the tunnel lines was found to be extremely disintegrated, no water was encountered—a favorable indication of the success of the recent grouting.

On January 5th, 1914, the work of removing the grout fill between the bulkheads commenced. A small drift was driven through the Astoria bulkhead, after which the door was raised and immediately placed in working order. The removal of the grout fill was then resumed, this work proceeding with very light blasting, toward the Bronx bulkhead door, which was raised on January 15th, 1914, and preparations were made for resuming the bench excavation—after a delay of 102 days, caused by the flooding of the tunnel.

In removing the grout fill, it was found to be in an unusual condition and stratification. Certain layers were extremely hard, and others were equally soft; some sections were highly stratified—others quite massive; some layers and masses of what might be described as solidified "laitance" were found, which never attained a greater hardness than that of the softest chalk, and having a curious chemical composition, entirely different from that of the original Portland cement from which it had been produced. Layers of sand also extended through the mass, and, in the vicinity of the water-hole, a 4-ft. blanket of coal, sand, shells, bits of wood, etc., was disclosed. No indications of water were apparent, other than the usual normal seepage through the concrete lining.

Several interesting conditions were encountered during the recovery of the flooded tunnel. Although large quantities of sand, coal, bits of wood, etc., were found, having been washed into the tunnel, and thus establishing proof of a direct connection with the river bed, no signs

of any fish were observed, though during previous heading flows many live fish, up to 8 in. in length, were washed in.

Of unusual interest is the fact that, though the sum total of cement injected from both ends, from the time of the flood to the completion of the grouting, was only 595 cu. yd., about 1 150 cu. yd. of solid grout were removed, in addition to 400 cu. yd. of sand, shells, coal, etc., to say nothing of the quantity of cement grout which remained in and filled the fissures themselves. A possible explanation of this is that in some manner the injected cement had mixed freely with very fine-grained decomposed dolomite, probably in sediment in the water, and that cement in the form of grout does not act as normal Portland cement in maintaining constant volume.

A peculiar disclosure was made on uncovering the end of the long grouting tube inserted from the Bronx end. The end of this tube was solidly embedded in a 12-in. cube of extremely hard grout, entirely surrounded by a 4-ft. layer of sand, coal, shells, etc., thus isolated from the bulk of the grout in the chamber between the bulkheads. Apparently, with each injection, the cement scoured away and was forced through this sand layer, which then resumed its normal form after each discharge.

In view of the recent flooding, it was resolved to exercise even greater precautions than heretofore. After careful consideration, a method of attack was devised which ultimately proved quite successful. This consisted of: (*a*) the construction of an inverted concrete arch over the remaining length of bench between bulkheads; (*b*) an even more thorough exploration of the underlying rocks with test holes as long as could be drilled by percussion drills; (*c*) the injection of cement through these holes in an effort to solidify the rock mass—the inverted arch acting as a resistance to this grouting; (*d*) the careful excavation of the exposed rock by very light blasting of small sections only; and (*e*) the immediate concreting of the tunnel lining on completing a short length of full section excavation.

The inverted arch was 24 in. thick and 8½ ft. below the concrete tunnel arch on the center line. As this working space did not permit of drilling to the tunnel invert extrados, the exploration was accomplished by first testing to a depth of from 5 to 6 ft. below the inverted arch. Drilling pits were then excavated and concrete-lined as an inverted arch, well bonded into the tunnel arch and the inverted arch

slab. Test drilling was then continued from these pits, thus reaching well outside of the necessary lines of excavation.

In removing this 122 ft. of bench, the entire rock encountered was extremely wet, and, although two flows of fair volume occurred, no unusual difficulties were experienced. This was undoubtedly due to the numerous injections of grout, which had been quite effective in consolidating the disintegrated rock structure. On uncovering the water-hole of October 5th, it was found to be a greensand seam, 24 in. wide, and of unknown depth, filled with sand, coal, shells, etc., through which a 9-in. layer of hard grout extended. No water was flowing, nor was there any further trouble from this fissure.

9.—Steel Lining.

Following the completion of the bench excavation, the erection of the cast-steel ring lining began. The installation of this lining was to secure impermeability through some 400 ft. of wet rock section extending on each side of the existing 50-ft. length of cast-iron rings. Being an after consideration, it was of necessity devised for its adaptability to the existing conditions. In contrast to the 50-ft. section of iron lined tunnel, the rock structure of the zone finally lined with steel rings was of such poor quality, and so treacherous and shattered, that it would have been extremely hazardous to increase the area of excavation necessary for the insertion of circular rings. Therefore, it was desired to maintain as small an excavated section as possible, and this resulted in the adoption of a metal lining of special design.

The usual assumption in designing a metal lining for subaqueous tunnel construction is that it is surrounded by a more or less plastic medium, and that no inward collapse can occur at any point without an outward distortion taking place at other points on the periphery; usually, the most suitable section to meet these conditions is a circular one. In this case, however, the tunnel being deeply embedded in rock and of a horse-shoe section, there were certain assumptions which could not be made in the case of a tunnel surrounded by soft material without cohesion. That is to say, the assumption on which the tunnel lining was designed was that, on the completion of the work, the spaces between the metal lining and the rock would be solidly filled with concrete or cement grout, which, when placed, would support

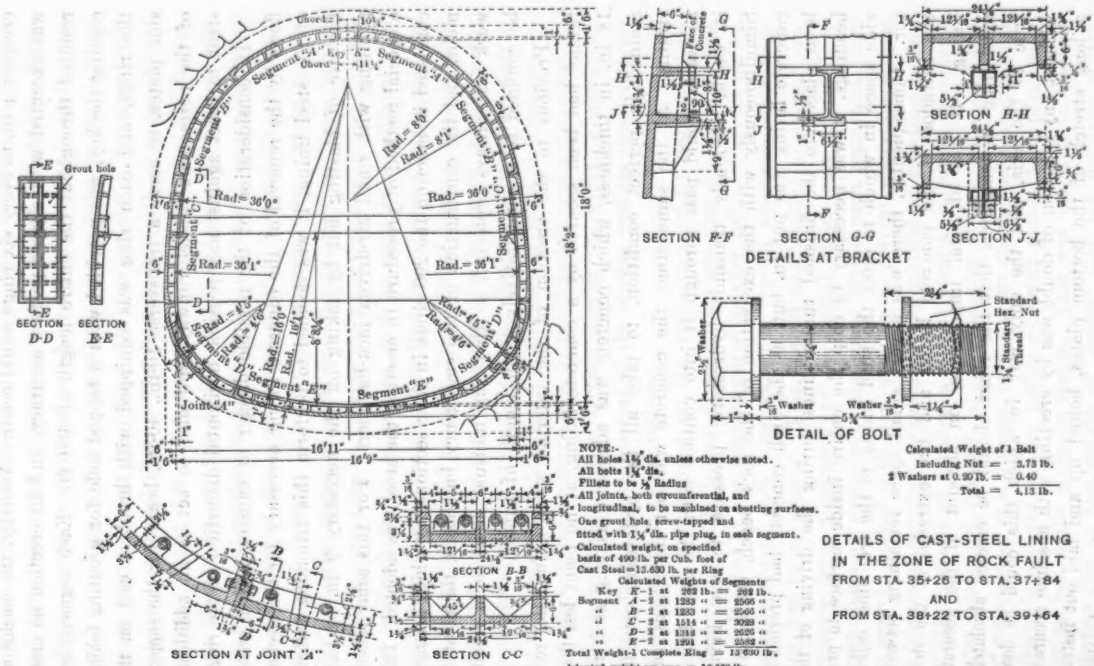
the entire metal lining and prevent any possibility of an outward distortion of the lining, the latter being designed with sufficient strength to prevent any inward collapse.

However, partly on account of the hydrostatic pressure, and partly on account of the departure from a true circular form, such stresses were produced in the metal that, without an extraordinarily wasteful design, cast iron could not be considered as a feasible material for withstanding the resultant pressures, and it appeared, therefore, to be necessary to substitute cast steel.

The cast-steel segments used for this purpose were made by the Wheeling Mold and Foundry Company, in accordance with the specifications of the American Society for Testing Materials. The segments were machined on all flanges to exact templates, so that all segments of the same letter were interchangeable. The facing of all joints of the steel segmental plates was laid out from the templates, which, at the same time, defined exactly the position of coupling bolt holes, which holes were drilled in the plate castings. Although the specifications permitted the use of cored bolt-holes, no such coring was used on account of the impossibility of obtaining the accurate fitting of segments with the irregularities which would have thereby been caused, the specifications requiring that this absolute accuracy of segment form in its relation to the bolt centers should be maintained.

This cast-steel lining, Fig. 16, was of segmental form, shaped to the neat lines of the tunnel, with the extrados 7 in. beyond these lines. As the flange depth of the rings was 6 in., and it was not desired to increase this depth at the expense of cutting away more of the exterior concrete lining, there was an allowance of only 1 in. of concrete over the ring flanges in covering the lower segments up to the springing line of the arch, being simply a protection against possible corrosion and to produce a continuously fair and smooth surface for the interior of the tunnel.

Each ring consisted of eleven segments, including the key; all joints were accurately machined. The key was placed at the top, and the joints were not staggered, as is the custom with circular iron tunnel rings. The rings were 24½ in. wide, this odd width being designed on the assumption that a slight excess in the gauge length of each ring would be probable, and that in erection a growth or



NOTE:-
All holes 1⅛" dia. unless otherwise noted.
All bolts 1¼" dia.
Fillets to be ½" Radius
All joints, both circumferential, and longitudinal, to be machined on abutting surfaces.
One grout hole, screw-tapped and fitted with 1¼" dia. pipe plug, in each segment.
Calculated weight, on specified basis of 490 lb. per Cub. foot of Cast Steel = 13,630 lb. per Ring

Calculated Weights of Segments

Key	$K-1$	at	262 lb.	= 262 lb.
Segment	$A-2$	at	1283 "	= 2566 "
"	$B-2$	at	1283 "	= 2566 "
"	$C-2$	at	1514 "	= 3028 "
"	$D-2$	at	1313 "	= 2626 "
"	$E-2$	at	1291 "	= 2582 "

Total Weight-1 Complete Ring = 13,630 lb.
Adopted weight per ring = 13,630 lb.

Calculated Weight of 1 Bolt
Including Nut = 3.73 lb.
2 Washers at 0.20 lb. = 0.40
Total = 4.13 lb.

DETAILS OF CAST-STEEL LINING IN THE ZONE OF ROCK FAULT FROM STA. 35+26 TO STA. 37+84 AND FROM STA. 38+22 TO STA. 39+64

FIG. 16.

creep of ¼ in. to every six rings would occur. Actually, the machining was executed with almost perfect accuracy, and in erection no creep resulted throughout the entire length of 400 ft. Each segment, excepting the key, was provided with a tapped hole for grouting behind the rings, and each ring was equipped with brackets cast on the side plates, to carry a transverse 10-in., 40-lb., beam for the support of the permanent tunnel runway, and also to act as an additional strut to react against any possible horizontal distortion of the rings.

In consideration of the possible danger of causing serious water flows by the removal of the concrete lining necessary for the erection of the steel lining, it was decided to perform this trimming without the use of blasting but by taking off the necessary 9 in. by bull-pointing with small air-driven jack-hammers. For this same reason, a special method of procedure in erecting the steel was adopted. This consisted in erecting the 200 rings in four sections, by which arrangement, and the construction of two emergency bulkheads, the point of work was always protected, so that any unusual flows could readily be confined to a comparatively small length of tunnel.

Previous to the erection of the internal steel lining, the rock tunnel had been secured by a concrete lining, at no point less than 18 in. in thickness, which concrete was adequate, as long as it remained in perfect condition, to take all the hydrostatic pressure strains. At the same time, the concrete was not impervious, and as water would pass through it into contact with the steel lining, it would obviously transmit hydrostatic pressure to that lining. Simultaneously with the excavation of the rock bench, the exterior concrete lining was put in, but, as the arch concrete had previously been placed to the finished tunnel lines during the driving of the headings, it was necessary to chip the entire inside surface to provide space in which to erect the steel lining to the true lines called for. Consequently, there was a very small space remaining between the intrados of the concrete lining and the extrados of the steel lining. It was of vital importance, on account of the accurate machining to templates, that the rings should be erected absolutely true to the axis, and the only way in which this could be done without any question of doubt was by erecting with absolute accuracy a long stretch of the bottom plates, bolted up and laid out instrumentally to perfect alignment and grade. In order to do this,

the bottom plates throughout were laid on a cradle consisting of three longitudinal strips of bar steel, 4 in. wide and ½ in. thick. These cradle bars were wedged up to proper alignment and grade, and secured by spiking with countersunk expansion bolts to the exterior concrete lining already in place. On this cradle were erected the two bottom segmental plates. The first short length of complete section lining, about 100 ft., was erected continuously on the bottom plates already set on the cradle, and each plate as erected was wedged with oak wedges to the exterior concrete lining already in place. The ends of this section were carefully secured by cementing, grouting, and packing, in order to maintain them rigidly and to plug the narrow spaces at each end, the theory then being to proceed with the grouting by leaving all grout holes open and pouring grout into each one in succession, and, as each hole indicated free flow of grout, to close each plug in its turn. It was thought that the plate lining, being held securely by hardwood wedges at each plate and the setting of the cement as grouting proceeded, would enable this section of steel to retain its form. Experience, however, showed that the hardwood wedges were inadequate to resist deformation in consequence of the pressures due to the flotation, and quite a serious distortion occurred in this section during the grouting operation, necessitating the entire removal of the section and its re-erection on the plan adopted later, as follows:

The rings were erected on the iron strip cradle previously described, on which the bottom plates for several ring lengths were first placed, followed by the same length of smaller radius side-wall plates. The remainder of each ring was then erected separately, this work being done by an erector. This method was adopted to prevent any divergence from alignment, as no special rings were provided to correct alignment deviation. The space behind each segmental joint was securely blocked with iron wedging to prevent distortion during the subsequent grouting operations. The method of grouting behind the rings consisted of forming a circumferential bulkhead of cement mortar behind two complete rings consisting of one whole and two half rings, spaced at a distance of five rings apart, and a longitudinal bulkhead behind the smaller radius side-wall plates, thus permitting grouting behind the rings in stages. This was essential to prevent distortion due to the weight of cement and the flotation of the

tunnel. To insure further against the possibility of distortion, the five-ring section between bulkheads was internally braced with heavy timbering (Fig. 15). Only one set of timbers was used, grouting operations being carried along in the rear of the erection at the same rate of progress in each. By this arrangement the five-ring sections were completely grouted before the removal of the timbering, the grouting being performed in three operations: first, the lower half above the longitudinal bulkhead; second, the upper half (thus keying in the top portion); and, third, the bottom plates. The first operation was with an air pressure of 30 lb.; the second and third were with 135 lb.

The rings were grommetted and caulked on the completion of the grouting. In grommetting, which was carried on immediately in advance of the caulking, each bolt was wrapped with strands of hemp, not coated with any water-proofing substance, as is usually the practice, as it was determined by experiment that the results derived from pitch-dipped hemp were not better than with the uncoated strands. In caulking the ring joints, small air-driven caulking hammers were used, the procedure consisting simply of turning over the metal edges of the joint, in every respect as though caulking a boiler. In unusually wet places lead wire was used to stop the flow temporarily until the edges were turned. This 400 ft. of steel lined tunnel is now practically water-tight, although occasional drops are caused by the natural sweating of the metal. This in time will probably gradually dry up and disappear entirely, due to the corrosion of the metal and the silting in of fine particles of matter.

10.—Drainage.

The completion of the tunnel with the entire lining left a seepage of water to be taken care of by the permanent drainage and the pumpage of 280 gal. per min., this seepage coming in from the entire length of the tunnel and the two shafts, including the enlargement at the foot of the Astoria Shaft. Consequently, this leakage was yielded by the equivalent of some 5 600 ft. of full-sized tunnel, or a surface area of approximately 36 000 sq. yd. This inflow for the most part finds its entry through the jointing of the rock in the hardest portions of the granitic gneiss, including the Astoria Shaft and the tunnel extending from that shaft to the dolomite rock con-

tact. The softer dolomite is, for all practical purposes, perfectly dry, and there is practically no water finding entry through the section lined with steel plates. The remainder of the inflow originates in the similar hard gneiss constituting the Bronx Shaft, and the section of the tunnel between the bulkhead line of the East River and the westerly contact.

The writer has invariably found that, so far as water flow through the rocks in the vicinity of New York is concerned, the greatest leakage is in the hardest rocks, and that wherever the mica schist or dolomite rocks exist, their softer character closes the joints and fissures to such an extent as to make these rocks practically water-tight.

Considerable efforts were spent on grouting through the lining to lessen the leakage from these hard rocks, including the drilling of long holes through the lining to intercept the minute joints and fissures, into which was forced cement grout under pressures of from 200 to 300 lb. per sq. in. For a time this would prove effective, but, within a few days after stopping the inflow, under these conditions, it would almost invariably be found that water would work its way through other channels, with the final result that the grouting of the rocks had little effect on the total quantity of water. There are indications, however, that in the course of time a great many of these minute fissures will choke themselves by the washing in of particles of solid matter to fill the crevices, a condition confirmed by observation of other tunnels.

11.—Ventilation.

The proper ventilation of the tunnel during the construction period was obtained with electrically-driven suction fans—one operated on the surface at each plant—capable of furnishing 2 500 cu. ft. of air per min. at a pressure of 5.4 oz. Each fan was at the top of each shaft, and was connected by a 12-in. inlet to an 18-in. pipe extended down the shaft and along the tunnel wall to within 200 ft. of the bench, sections being added as the advance proceeded. The fans were not operated continuously, but only until proper ventilation was secured after each blasting operation. On the Astoria side a 30-in. blower, driven by a compressed-air engine, was erected in the tunnel 1 700 ft. from the shaft, as the 12-in. fan was not efficient beyond this length. This blower was connected to the existing 18-in.

line from the 12-in. fan, the fan being discontinued when the blower was put in operation. In tunneling through the zone of decomposed dolomite, the construction of bulkheads close to the point of work in the heading naturally interfered with proper fresh air circulation, and at times the inflowing water carried with it sludge tar which had probably been discharged many years previously from adjacent gas-works and had been lying on the bed of the river in deep holes. This material gave off fumes which were unpleasant and caused a burning sensation in the eyes, but at no time during the work was there any illness or inconvenience attributable to improper ventilation.

12.—Lighting.

The tunnel was lighted electrically by current supplied at each plant by two direct-current generators, each of 250 volts, 200 amperes, and 50 kw. These were operated alternately, except at the Astoria end, where it was necessary to operate both generators at night after the tunnel had been driven 1 000 ft.

Lamps of 16 c.p. were used while in the heading and bench. Luminous-arc headlights were carried along with the advance, the result being very satisfactory, as these lamps gave a bright whitish light, quite essential through the dark, wet, rock sections, and especially in driving through some dense, black gneiss encountered. After entering the decomposed, water-bearing zone, kerosene hand-lanterns were used throughout the tunnel as a measure of safety, and were kept lighted at all times in case of emergency.

13.—Triangulation and Alignment.

The triangulation system of the Astoria Tunnel was first laid out in 1903-04, and conformed with the intentions at that time of constructing the tunnel to 111th Street, Manhattan. Subsequently, this triangulation system, with the necessary modifications, was adopted for the alignment of the present Astoria Tunnel.

The triangulation system of the present tunnel consisted of three scalene triangles constructed on an 1 800-ft. base line on the Astoria shore, with a 1 300-ft. check base on the Bronx shore, one angle point being on the extreme northerly end of the "Sunken Meadows" in the East River. The accuracy of the triangulation work was proved by the fact that the computed length of the Bronx base checked to within 0.006 ft. of the measured length, or 1 in 218 000.

The alignment was transferred down the shafts of the tunnel with suspended plumb lines which were aligned by direct sight across the river between shafts, triangulation towers being provided for this purpose. Some little difficulty was caused by the oscillation of the wires due to eddying air currents and the water seepage down the shaft sides, these conditions necessitating the use of 35-lb. wing weights. At the Astoria Shaft the base between the suspended wires was 28 ft.; in the smaller Bronx Shaft, the base was only 21 ft.

In the extension of the tunnel center line, all fixed points were placed in the concrete arch—those in the Astoria heading were at 300-ft. intervals, but in the Bronx heading a maximum of only 200 ft. could be used, on account of the continual fog. As the headings approached the water-bearing rock section, it was found necessary to reduce the sight lengths considerably, especially after the construction of bulkheads, as the fog then became, in a sense, confined. All instrument work was performed from suspended transit tables and scaffolds, thus preventing any delays to, or confusion between, the engineering and construction forces.

Accurate elevations were determined from a bench-mark on the northern shore of the Bronx Kills opposite Randalls Island. The reference of this bench-mark to a mean-sea-level datum plane was supplied to the tunnel engineers by the United States Coast and Geodetic Survey, being based on the result of 6 years of tidal observations at Sandy Hook, N. J. The location of this bench-mark necessitated a level run of 3 900 ft. to the Bronx Shaft and a run of 14 000 ft. to the Astoria Shaft, the latter route crossing Randalls and Wards Islands to reach the point of minimum width of the East Channel of the East River. Thus, the length of the combined transfer of elevations of 3½ miles on the surface to the point of closure in the tunnel was approximately 4½ miles, in view of which the error of only ⅛ in. is quite remarkable.

In consideration of the unfavorable conditions under which the underground work was necessarily performed, the meeting of the alignment and grades of the two headings is regarded as most satisfactory. As mentioned, when the tunnel entered the water-bearing rock, there was considerable difficulty in obtaining even fairly distinct sights, as there was a continual fog caused by the condensation of the air exhaust from the drills, pumps, and grout pans. The alignment,

which was carried 3 900 ft. from the Astoria Shaft and 762 ft. from the Bronx Shaft, met to within 1¼ in. for line, ⅛ in. for grade, and ½ in. for stationing.

14.—Quantities and Costs.

Table 7 is a summation of the quantities incidental to the shaft and tunnel construction.

Table 7.—Summation of Quantities.

Earth excavation, for erecting plant	1 750	cu. yd.
Earth excavation, shafts	3 050	"
Earth excavation, total	4 800	"
Rock excavation, shafts	17 480	"
Rock excavation, tunnel	70 140	"
Rock excavation, total	87 620	"
Concrete removal, tunnel	4 080	"
Grout bank removal, tunnel	1 150	"
Summary of excavation—earth, rock, concrete, etc	97 650	"
Concrete, plant foundations	1 160	"
Concrete, shaft lining	5 070	"
Concrete, tunnel	26 860	"
Concrete, tunnel bulkheads	2 070	"
Concrete, pipe foundations	710	"
Summary of concrete placed	35 870	"
Lining rings, 20 cast-iron, 50 lin. ft	227½	tons*
Lining rings, 200 cast-steel, 400 lin. ft	1 365	"
Lining rings, total, 220 rings, 450 lin. ft	1 592½	"
Cement, concrete plant foundations	6 870	bags
Cement, concrete lining, shafts and tunnel	199 000	"
Cement, concrete bulkheads	16 160	"
Cement, concrete pipe foundations	4 590	"
Cement, total in concrete	226 620	"
Cement, grouted in shaft lining	1 200	"
Cement, grouted in tunnel concrete lining	45 340	"
Cement, grouted in metal lining	14 380	"
Cement, grouted in bulkheads	26 510	"
Cement, grouted in test holes	28 550	"
Cement, grouted in pipe supports	2 000	"
Cement, total in grout	117 980	"
Cement, total in concrete and grout	344 600	"

*Tons of 2 000 lb.

In this undertaking the cost of construction must be classed in two accounts: first, the cost of the shafts and tunnel constructed under normal conditions in the two types of rock formation; and, second, the extraordinary cost and expense of that portion of tunnel, approximately 400 lin. ft., constructed in the decomposed rock formation. The cost of the permanent pipes and interior equipment is an additional and entirely distinct item of expense, bearing no direct relation to the cost of the structure of the tunnel.

It is worthy of note that the unit costs, based on all work of a normal character, with the sole exception of the great expense of getting through the decomposed rock, was less than the originally estimated figures contemplated at the commencement of the work.

The unusual conditions involved in that portion of the tunnel driven through the decomposed rock formation, and the long delay in conquering the difficulties which arose, involved an actual cost for that section materially greater than had originally been allowed.

Tables 8 and 9, showing the distributed unit costs for the shaft and normal tunnel constructions, are of interest and value, and show advantageous results under the conditions governing the work. The unit costs given in these tables are the totals for both labor and material, allowing 60% of the purchase value of plant and equipment —the remainder being considered as a credit for ultimate salvage. The item "Contingencies" includes all expenses chargeable to accidents, damages, hospital, rentals, insurance, and operation of the ferry-boat used between the shafts.

TABLE 8.—UNIT COSTS OF SHAFT CONSTRUCTION—NORMAL CONDITIONS.

Items.	Cost per Actual Cubic Yard.			
	Rock excavation (Gneiss).		Concrete Lining.	
	Astoria Shaft.	Bronx Shaft.	Astoria Shaft.	Bronx Shaft.
General supervision	$0.114	$0.120	$0.130	$0.120
Engineering expense	0.193	0.204	0.221	0.205
Field office administration	0.808	0.853	0.922	0.855
Plant and equipment, installation and maintenance	0.346	0.365	0.396	0.366
Plant and equipment, 60% of purchase value.	0.867	0.915	0.990	0.917
General surface labor and supplies	0.354	0.373	0.451	0.375
Drilling and blasting	1.971	1.571	1.083	1.283
Mucking	1.652	1.797	.....	.....
Disposal, surface transportation	0.484	1.592	.....	.....
Disposal, scow line and towing	0.149	0.101	.....	.....
Power and plant running	1.625	1.343	.....	.....
Making and sharpening steels, and repairing machines	0.420	0.376	.....	.....
Lighting	0.157	0.171	0.134	0.161
Extending lines (air, pump, etc.)	0.173	0.124	0.100	0.098
Pumping	0.301	0.439	0.234	0.207
Contingencies	0.817	0.879	1.113	1.078
Forms and platforms, making, setting and removing	.....	.....	2.389	1.902
Use and waste of lumber	.....	.....	0.300	0.294
Handling materials	.....	.....	0.519	0.341
Sand	.....	.....	0.264	0.424
Cement	.....	.....	1.898	1.807
Stone	.....	.....	0.836	0.806
Mixing concrete	.....	.....	0.784	0.779
Transportation	.....	.....	0.055	0.000
Placing concrete	.....	.....	1.342	1.650
Cleaning up	.....	.....	0.045	0.091
Totals	$10.431	$11.223	$14.206	$13.759

TABLE 9.—UNIT COSTS OF TUNNEL CONSTRUCTION—NORMAL CONDITIONS.

Items.	Cost per Actual Cubic Yard.		
	Rock excavation.		Concrete lining.
	Gneiss.	Dolomite.	
General supervision	$0.087	$0.069	$0.064
Engineering expense	0.151	0.121	0.127
Field office administration	0.629	0.503	0.464
Plant and equipment, installation and maintenance	0.283	0.226	0.209
Plant and equipment, 60% of purchase value	0.662	0.529	0.488
General surface labor and supplies	0.327	0.261	0.326
Drilling and blasting	1.997	1.672	.....
Timbering	0.005	0.001	.....
Mucking	1.736	1.213	.....
Disposal, tunnel and surface transportation	0.795	0.748	.....
Disposal, scow line and towing	0.279	0.249	.....
Power and plant running	1.262	1.055	1.256
Making and sharpening steels, and repairing machines	0.285	0.306	.....
Lighting	0.158	0.152	0.183
Extending lines	0.139	0.142	0.161
Pumping	0.152	0.116	0.147
Contingencies	0.761	0.626	0.984
Wood forms, making, setting and removing	.....	.....	0.857
Steel forms, making, setting and removing	.....	.....	0.125
Steel forms, charges	.....	.....	0.353
Use and waste of lumber	.....	.....	0.187
Handling materials	.....	.....	0.332
Sand	.....	.....	0.323
Cement	.....	.....	1.918
Stone	.....	.....	0.891
Mixing concrete	.....	.....	0.487
Transportation	.....	.....	0.702
Placing concrete	.....	.....	1.709
Cleaning up	.....	.....	0.270
Totals	$9.708	$7.989	$12.563

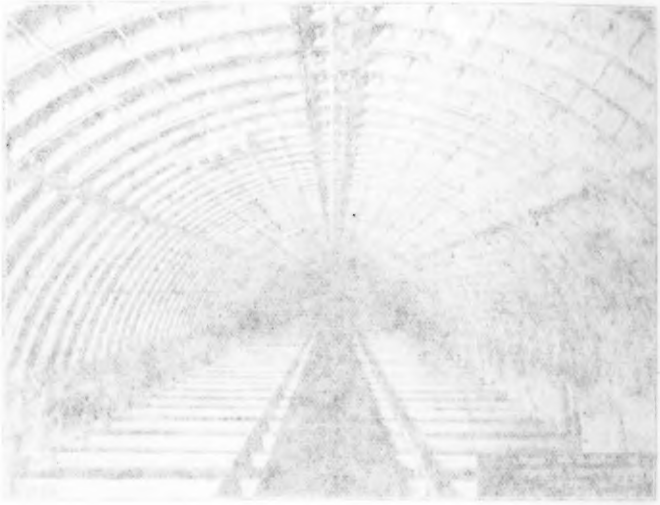
15.—CAST-IRON PIPES IN THE TUNNEL.

The cast-iron pipes in the tunnel from Astoria to The Bronx are the largest ever made. Their internal diameter is 72 in.; the thickness of metal in the pipe walls is 2¾ in.; they have hub and spigot joints, and were cast in standard 12-ft. lengths. The weight of each pipe is 26 000 lb., and the displacement of water by each length represents some 24 800 lb., from which it is seen that, even should the tunnel be allowed to fill with water, the pipes would retain their position secure.

These mains are arranged in two parallel lines occupying the lower portion of the tunnel, and are supported on concrete bed-blocks. The structural steel runway, Fig. 17, for supporting any future utilities that may be put in, extends through the tunnel and is just above the mains. The concrete blocks are 6 ft. apart, each block supporting

FIG. 17.—TUNNEL RUNWAY THROUGH CAST-STEEL RING SECTION.

FIG. 18.—LOWERING 72-IN. CAST-IRON T DOWN SHAFT.



both lines of pipe. Each 12-ft. length of pipe is supported by two blocks, and the joints of the two pipe lines are staggered. Every second block supports a cast-iron column which braces the center of the transverse runway beams, the actual support for the runway being provided by cast-iron brackets anchored to the concrete tunnel walls. In both shafts the two 72-in. pipe lines are connected to the tunnel lines by double-spigot eccentric reducers and bell-spigot bell tees. All the vertical risers are 12 ft. long, with both the spigots and bells machined on the face to take the true bearing of the vertical pressure loads. At the top of each shaft flanged spigot tees, reducing to 48 in. diameter, connect to 48-in. pipes through the shaft lining into concrete valve pits, thence passing underground—in Astoria to the gas holders, and in The Bronx to the street feeder lines. Each shaft is equipped with an elevator and an emergency ladder, the elevator framing also acting as a bracing for the pipe lines. The tunnel runway extends to the elevator landing at the bottom of each shaft, and in the Astoria Shaft there is an operating floor in the enlargement.

During the driving of the tunnel, arrangements proceeded for putting in the permanent piping, and consisted of unloading and storing the pipe and structural steel, casting the concrete pipe bed-blocks, setting bolts in the tunnel side-walls for the support of the tunnel runway, and the design and construction of additional plant equipment necessary in laying the pipe.

The pipes, excepting those for the Bronx Shaft, were stored at the Astoria plant, as the required space was available there and as it was also the intention to lay them from the Astoria side, this shaft affording the larger working area. The concrete blocks were cast on the surface at the Astoria plant, the work being completed fully a year before the pipe-laying commenced, so as to insure the maximum strength. Collapsible wooden moulds were used, and the mix was a rich gravel concrete, as it was desired to secure a thoroughly sound block. The blocks contained reinforcing rods, bolts for securing the runway columns, and small pipe holes to which the slugs were attached when lowering into the tunnel. As the saddles were long and of irregular shape, the reinforcing rods were necessary to prevent fracture due to their dead load when lifted by the derrick.

The permanent bracket bolts for the support of the tunnel runway are 12 ft. apart, and there are two bolts at each bracket; but, as it was

intended to use a traveling crane, running on steel girders secured to the tunnel walls, for which two bolts were insufficient, advantage was taken of the necessity of the permanent bracket bolts and the crane track design arranged to utilize those bolts. This resulted in placing four bolts at each bracket location. The bolts were 1¼ in. in diameter, 24 in. long, and were secured 18 in. in the concrete side-walls.

In laying out the bracket holes, the marking points were permanently secured, each consisting of a small nail driven into a wooden plug set in the concrete walls of the tunnel; thus the points were preserved until a year later, when they were used for spacing and grading the pipes. This simplified the necessary field engineering during the laying of the mains, and was a decided factor in the excellent progress attained.

The principal additions to the plant equipment, incidental to the laying of the 72-in. mains, consisted of a 50-ton steel derrick and engine, a traveling crane, and two pipe-testing machines.

The steel derrick for handling the 72-in. mains, Fig. 18, had a 50-ft. boom and a 35-ft. mast. Power was supplied by a steam-driven engine, having a 10-in. cylinder and a 13-in. stroke. The swinging gear was operated independently by a rotary engine. The hoisting rope was a six-part line of ¾-in. special wire cable, and the boom fall was a twelve-part line of ¾-in. wire cable. In operating this engine, the load was lowered by throwing the clutch into gear and allowing the engine to run backward, instead of throwing on the brakes, as the friction burned up the wooden brake bands too rapidly. This method was considerably safer, and gave better control, than by lowering with a brake. Special slings were constructed for handling one pipe and two saddles together.

The traveling crane, Figs. 19, 20, and 21—designed by Mr. Hodgson for handling and erecting the pipe within the tunnel—was constructed on a wood frame fitted with cast-steel wheels running on steel girders attached to the concrete tunnel walls by the 1¼-in. bolts previously mentioned. The wheel base of the crane was the same as the bracket spacing, so that the load on the bracket could not exceed the maximum wheel load of the crane, which was 15 000 lb. The crane was operated by two 5-h.p. air engines—one for the longitudinal and the other for the transverse movement. The speed was from 40 to 50 ft. per min., quite sufficient for the short distance covered. Each end of the crane

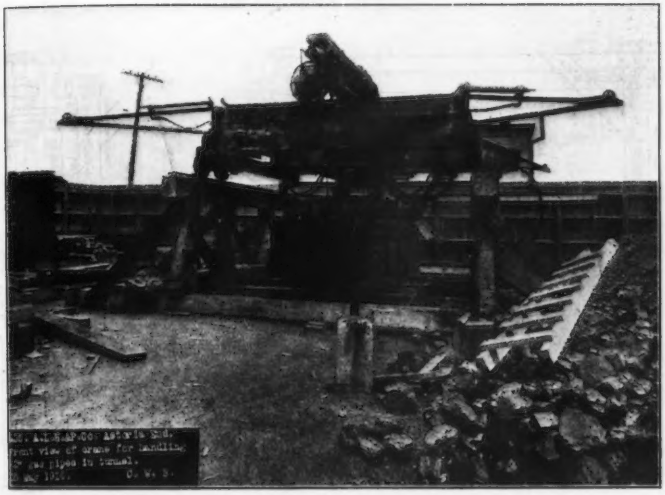
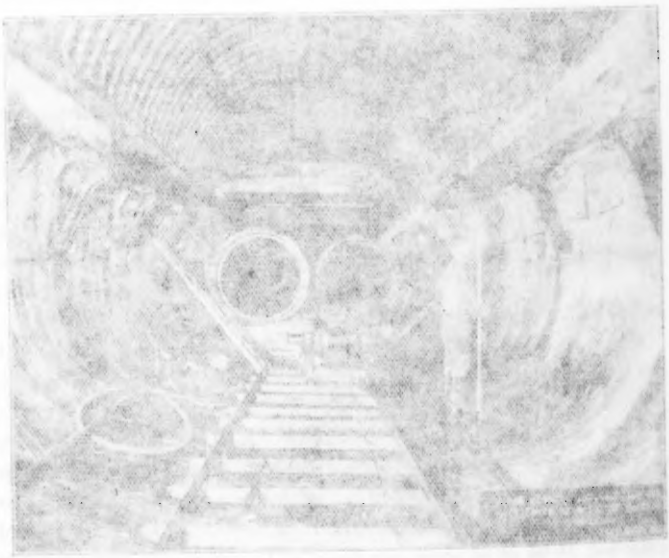
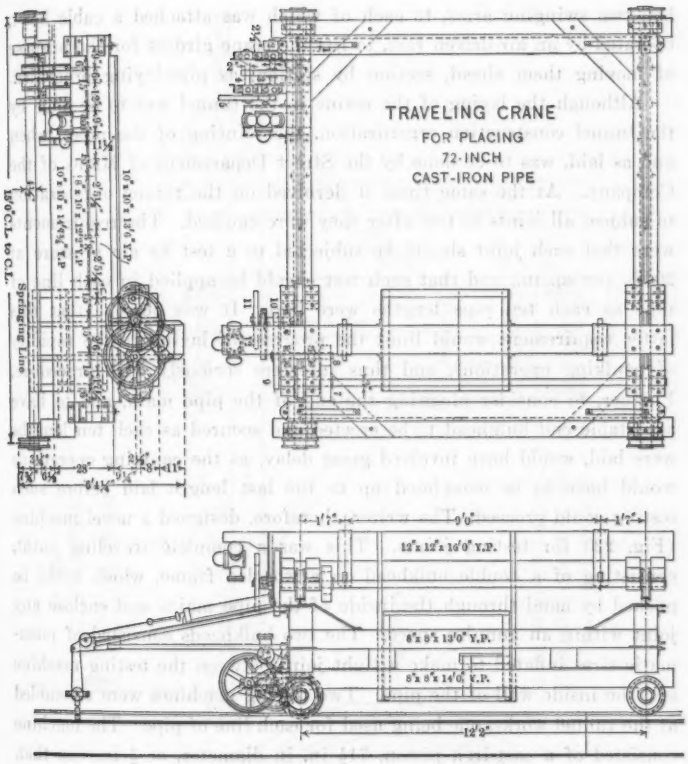


FIG. 19.—CRANE FOR HANDLING 72-IN. CAST-IRON PIPES IN TUNNEL.

FIG. 20.—SETTING 72-IN. PIPE IN STEEL RING SECTION OF TUNNEL.





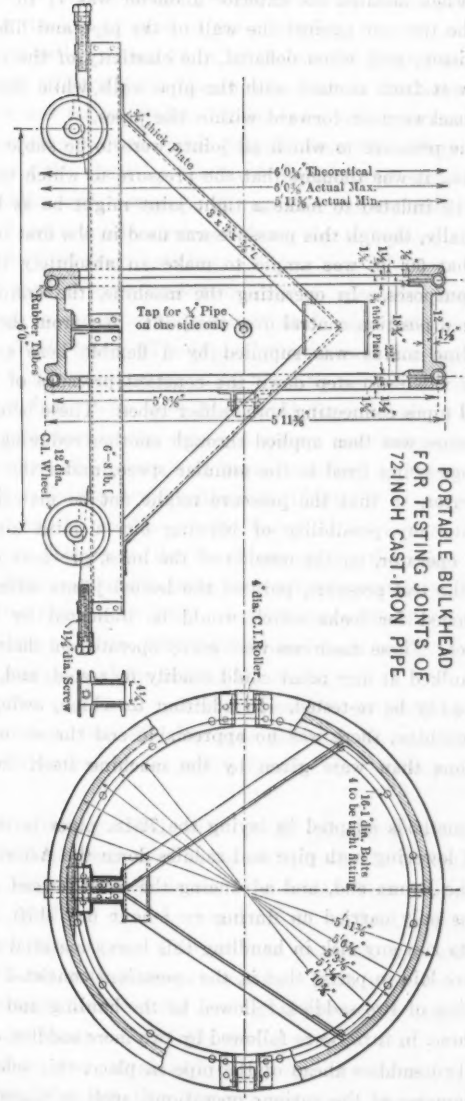
DIMENSIONS OF GEAR WHEELS

Marked	Pitch diameter	Pitch	Number of Teeth	Width of Face	Diameter of Hole	Size of Key way	Material	Hub
1	17.188"	1⅛" Circular	36	4"	4 23/64"	¾"	C.I.	4¼"
2	6.207"	1⅛" "	13	4"	2½"	¾"	Steel	4"
3	17.188"	1" "	54	2½"	2½"	"	C.I.	3"
4	5.092"	1" "	16	2½"	2"	½"	Steel	"
5	24"	4" Diametral	96	1¾"	2"	"	C.I.	"
6	4"	4" "	16	2"	1 15/16"	¾"	Steel	2"
7	14.483"	1¾ Circular	26	4"	2 37/64"	¾"	C.I.	4¼"
8	7.798"	1¾" "	14	4"	2½"	¾"	Steel	4"
9	26.739"	1" "	84	2½"	2½"	"	C.I.	2¾"
10	4.456"	1" "	14	2¼"	2"	½"	Steel	2⅛"
11	24"	4" Diametral	96	1¾"	2"	"	C.I.	3"
12	4"	4" "	16	2"	1 15/16"	¾"	Steel	2"

FIG. 21.

had two swinging arms, to each of which was attached a cable hoist, operated by an air-driven ram, to lift the crane girders for the purpose of moving them ahead, section by section, as pipe-laying proceeded.

Although the laying of the mains in the tunnel was to be done by the tunnel construction organization, the jointing of the pipes, when and as laid, was to be done by the Street Department of Mains of the Company. At the same time, it devolved on the tunnel organization to subject all joints to test after they were caulked. The requirements were that each joint should be subjected to a test by air pressure at 20 lb. per sq. in., and that such test should be applied in each line of pipe as each ten pipe lengths were laid. It was obvious that this latter requirement would limit the progress of laying to the rapidity of caulking operations, and thus interfere seriously with procedure. Further, to consider plugging the end of the pipe main, and to have a portable end bulkhead to be erected and secured as each ten lengths were laid, would have involved great delay, as the caulking operations would have to be completed up to the last length laid before such testing could proceed. The writer, therefore, designed a novel machine (Fig. 22) for testing joints. This was a complete traveling outfit, consisting of a double bulkhead on a wheeled frame, which could be pushed by hand through the inside of the pipe mains and enclose any joint within an annular space. The two bulkheads consisted of pneumatic tires inflated to make a tight joint between the testing machine and the inside wall of the pipe. Two testing machines were assembled at the tunnel works, one being used for each line of pipe. The machine consisted of a cast-iron piston, 71½ in. in diameter, or ½ in. less than the internal diameter of the pipes. Previous to laying, a great number of pipes had been calibrated, and the internal diameter was found to range from a minimum of 71⅝ in. to a maximum of 72¼ in. This piston was 24 in. long, with exterior flanges, giving an annular space between flanges of 12 in. The two flanges were designed of cove form, to give close support to a soft and elastic rubber tube, and the exterior rings, attached by bolts to the center casting, like a junk ring to a piston, enabled the rubber packing rings to be inserted and, when inflated, to give the necessary support. These flanged grooves were fitted with a solid continuous tire tube, made of the finest inner tube rubber, and fitted with a standard automobile tire valve, the tube being 2½ in. in external diameter with ⅜-in. rubber walls. These tubes were made up



PORTABLE BULKHEAD FOR TESTING JOINTS OF 72-INCH CAST-IRON PIPE

Fig. 22.

so that when deflated the exterior diameter was 71 in. The inflation forced the tire out against the wall of the pipe and filled the grooves of the piston, and, when deflated, the elasticity of the rubber entirely withdrew it from contact with the pipe walls, while the machine was pushed backward or forward within the pipes.

As the pressure to which all joints were to be subjected was 20 lb. per sq. in., it was thought that the pressure to which the tubes would need to be inflated to make a tight joint might be as high as 80 lb., but, actually, though this pressure was used in the first instance, it was found that 50 lb. was ample to make an absolutely tight joint for testing purposes. In operating the machine, the annular space was moved to a position central over the joint. Air from the high-pressure tunnel line mains was supplied by a flexible hose and, through a reducing valve—to step down the constant pressure of 50 lb.—to the manifold pipes connecting both rubber tubes. These being inflated, the test pressure was then applied through another reducing valve, a pressure gauge being fixed to the annular space, under the observation of the operator, so that the pressure might not be materially exceeded, to obviate any possibility of blowing out a joint already caulked. Another operator, on the outside of the hubs, previous to the application of the test pressure, painted the leaded joints with soap solution and watched for leaks which would be indicated by the formation of bubbles. These machines were easily operated on their roller frames, joints caulked at any point could readily be tested, and, if re-caulked, could readily be re-tested. In addition to which, owing to the form of the machine, there was no appreciable end thrust to require other precautions than were given by the machine itself to balance that thrust.

The methods adopted in laying the 72-in. pipes in the tunnel consisted of lowering both pipe and saddles down the Astoria Shaft, hauling to the Bronx end, and advancing the work toward Astoria. This work was only carried on during an 8-hour day shift, as it was not desired to run any risk in handling this heavy material at night. The pipes were laid in pairs; that is, the operations consisted of the hauling and setting of two saddles, followed by the hauling and setting of two pipes; these, in turn, were followed by two more saddles, etc. By maintaining two saddles ahead of the pipe in place, this scheme permitted an overlapping of the various operations, such as placing and setting

saddles, setting pipe, moving crane girders, removing hauling track, etc., thus reducing the loss of time to the minimum.

The pipes, which varied within 1½ in. of the correct length, were all numbered, and were selected from the storage rows as the work proceeded. Each pipe was transported to the shaft by rolling it on skids by a single cable engine with the cable looped around it. They were lowered singly down the shaft by the steel derrick, and with every other pipe two saddles were lowered. A tag line was attached from the pipe to a steel cable guide stretched from top to bottom of the shaft, to prevent the load from twisting.

The pipes and saddles were landed directly on specially designed flatcars, and haulage to and from the point of work was by the endless cable system on a single track. The pipe cars were run at a speed of 6 miles per hour and the other loads at 10 miles per hour. These speeds were greater than those usually adopted for an endless cable system in tunnel work, but in this case the system was designed with the minimum number of sheaves and turns possible, so that these speeds were readily maintained and without accident.

On the arrival of the saddles at the point of work they were transferred from the flatcars to the tunnel invert by the crane and accurately set to alignment, grade being maintained by steel wedging beneath them. The pipes were taken directly from the flatcars to position by the crane, two 1-in. wire cable slings, each attached to a 10-ton chain block, being used for raising the loads.

The setting of the pipes was followed, some 200 ft. back, by the caulking, and this at a like distance was followed by the testing machines; so that there was no confusion between the various gangs engaged.

The joints were hand-caulked with small air-driven caulking hammers, hemp yarn and lead wool being well driven to a depth of 5½ in. into the caulking space, which was ⅝ in. wide. This required 4 lb. of soaped yarn, 10 lb. of dry yarn, and 225 lb. of lead, per joint, the latter including 2% waste. The average progress was sixteen joints caulked each day with sixteen hammers. Each pipe line was provided with a portable bulkhead for testing, as previously described. After this testing, the joints were thickly coated with red lead.

Excellent progress was attained in laying the tunnel pipe, especially considering the fact that the entire tunnel transportation

in both directions was over a single track. An average daily progress of twelve pipe lengths was attained for the whole work, and in the half tunnel length last executed the average progress was as great as sixteen lengths per day.

The erection of the structural steel runway proceeded from the Bronx end toward the Astoria Shaft, the material being lowered down the Bronx Shaft. No difficulty was found in doing this work at the same rate as pipe-laying, and it was not featured by any incident of unusual interest.

Personnel.—The undertaking herein described was carried out for the Astoria Light, Heat and Power Company (an organization closely affiliated with the Consolidated Gas Company of New York), and was under the general supervision and direction of Mr. William H. Bradley, Chief Engineer, and William Cullen Morris, M. Am. Soc. C. E., Engineer of Construction. Mr. Colin C. Simpson, Superintendent of Mains, had direct charge of jointing the pipes inside the tunnel and of laying the street mains connecting therewith.

The development of the plan for the project of building the Astoria Gas Plant and the Astoria Tunnel herein described, must be attributed directly to Mr. Bradley.

The design, engineering, and construction of this entire work were executed by Jacobs and Davies, Incorporated, as Tunnel Engineers, for whom the writer acted as Engineer-in-Charge. The field engineering, including surveying, alignment, and detailed field work in connection with construction, was first in charge of Mr. G. F. Weismann, and during the later period was in charge of Mr. J. Chadwick Scott, who had previously acted as Assistant Field Engineer. The construction of the work, throughout, was in charge of Mr. Vivian Messiter as Engineers' Agent, assisted by Mr. Alexander W. Hodgson as Mechanical Engineer.

DISCUSSION

F. LAVIS,* M. AM. SOC. C. E. (by letter).—During the past few years there has been a notable development, and almost entirely in the vicinity of New York City, in the construction of tunnels at such depths below water level as to preclude the possibility of using air pressure to prevent the inflow of water, should seams in the rock afford it an opportunity to reach the tunnel with the full head due to the depth below the water surface. Mr. Lavis.

The other tunnels of this nature which have been built, have been along the line of the Catskill Aqueduct, where the restrictions confining the location within certain definite limits were not so rigid as in the case of the Astoria Tunnel, and, therefore, it was possible to fix them with greater regard for the avoidance of probable bad geological conditions. In the case of the deep siphons of the Aqueduct, sufficient diamond drill borings were taken to enable competent geologists to predict fairly successfully the character of the rock, and the location was made in such a way that the tunnel would lie wholly within the zone in which it was expected to find sound rock, free from faults, or to cross the latter at or nearly at right angles. So far as the writer knows, these expectations were reasonably well fulfilled, and although grouting of water-bearing seams was resorted to, and very high pressures were used, there has not been recorded, thus far, any experience comparable to that encountered in the Astoria Tunnel, and so well and fully described in this paper. The latter is unique in describing methods of driving a tunnel through unstable ground with seams of decomposed rock having direct connection with navigable waters, at a depth of more than 100 ft. below the lowest depth at which work could be carried on under air pressure, the recovery of the tunnel after complete flooding, and the solid grouting of the whole tunnel section to form a stable material through which the tunnel was finally driven.

It seems to the writer that it is out of the question to discuss in much detail the methods described in the paper; we have had no similar experience with which to compare it—the situation was unique, the methods were effective, and the job is finished. Some other scheme might have been effective, but no one can say positively that it would, the only possibility known to the writer is freezing, and there is nothing in past experience to lead one to believe that it would have been better, or so good. Probably it is only effective in fairly homogeneous material softer than rock, and probably would not work at all in the shattered rock encountered in this tunnel.

The Society, as well as the author, is to be congratulated on the excellent manner in which the paper has been presented, and the

* New York City.

Mr. Lavis.

completeness of the description contained therein. The writer believes that his paper,* describing the construction of the Bergen Hill Tunnels of the Pennsylvania Railroad, contained the first tabulated statements in reference to any tunneling work in North America, showing number of feet drilled per cubic yard, quantities of explosives, actual drilling time, etc., though, in a lesser degree, such records had been published in regard to some of the Alpine tunnel work in Europe. He has noted, therefore, with considerable satisfaction, the presentation of similar data, not only in this paper, but in several other descriptions of tunneling operations in recent years. The statement of progress in number of cubic yards per shift or per day is far more definite than the statement of the number of feet of advance which may or may not be of the full clean section. The statement showing the complete items of the plant is most useful, as are also the details, so often omitted, of the smaller items, in regard to the excavation of the rock, such as quality of steel, sizes of holes, kind of fuse, batteries, etc.

The construction of the concrete arch to hold the roof in bad ground before building the side-walls or invert is not new in this tunnel, although it is a comparatively recent development in the art of soft-ground tunneling. So far as the writer knows, its first important use was in the Providence Tunnel of the New York, New Haven and Hartford Railroad, built about 10 years ago.

The question as to the desirability or otherwise of using dry rock packing to fill the space between the normal section of the lining and the actual excavation is an interesting one. In this case, as well as in nearly all cases of the tunnels on the Catskill Aqueduct, tight concrete packing and thorough grouting were clearly indicated as the only effective method of surely preventing the least disturbance by settlement of the surrounding ground. In cases similar to that of the Bergen Hill Tunnels of the Pennsylvania Railroad, however, the use of dry rock packing over the arch was not only far less expensive, but proved to be most effective in allowing the ground-water to reach the drainage pipes at the sides and thus enter the side drains in the floor, providing a dry tunnel with very little water-proofing.

The value of this paper and the importance of the work which it describes can only be fully understood by a realization of the fact that it marks a distinct step forward in the art of tunneling. For ages, almost, tunneling was only possible through fairly sound rock, or materials which could be held by close timbering. The shield and the use of compressed air, brought into really general use only within the last 30 years, although they had been used previous to that time, enable tunnels to be driven through water-bearing materials to depths of about 100 ft. below the surface of the water, the pressure at greater depths being so great as practically to prohibit effective work. We

* *Transactions*, Am. Soc. C. E., Vol. LXVIII, p. 84.

now have a method of driving tunnels at almost any depth in water-bearing ground, and, within certain limits, through almost any class of material. Mr. Lavis.

Perhaps it is not without interest at this time, when it has been suggested that mining engineers should be called in to advise as to methods of subway construction, to note—without in the least detracting from the ability of mining engineers and miners in their particular field—that nearly all the developments in the art of tunneling through difficult ground, beyond the art of timbering in comparatively small sections, have been made by civil engineers, and for transportation purposes, rather than by mining engineers or miners. In the really vast tunneling operations which have been carried on in New York City and its vicinity in recent years, it is undoubtedly within the bounds of truth to state that not only has this work been the greatest, most extensive, and most difficult of its kind, that is, of tunneling, ever undertaken and carried out successfully anywhere, but it has been done with fewer accidents, and all the engineers responsible for this state of affairs deserve commendation.

The record of this work on the Astoria Tunnel shows, also, as do all records of successful advances in the art of tunneling, and nearly all records in recent years of advance in the rate of progress, the absolute necessity of such preliminary study, careful planning, and efficient organization as usually only engineers are trained to carry out. It must be remembered, however, that though a carefully studied plan must be worked out and adhered to, engineers engaged in this kind of work must be able to adapt it to all sorts of variations as they arise. It is far from easy, amid the noise, dirt, partial darkness, and, oftentimes, apparent confusion underground, to carry out a plan which looks well on a drawing-board in a comfortable, well-lighted office; and when added to all this, the whole East River is likely to drop in on one at any minute, plans are likely to go by the board.

MILTON H. FREEMAN,* ASSOC. M. AM. SOC. C. E. (by letter).—One of the most interesting features of this paper is the description of the method of excavating through decomposed water-bearing rock. An excellent opportunity was afforded to obtain positive data concerning the results of the grouting, as the subsequent excavation exposed the grout-filled seams. In work of a similar character on the Catskill Aqueduct, the following were found to be essential features: Mr. Freeman.

1.—An impervious bulkhead against which to grout, which also furnishes an anchorage for the grout pipes;

2.—A grout tank pressure considerably above the rock water pressure against which grout is injected;

* Brooklyn, N. Y.

Mr. Freeman.

3.—Adapting the consistency of the grout to the size of the cavity to be filled, using thicker grout for very large seams, and very thin grout for the fine seams.

On the Catskill Aqueduct the first attempt to fill water-bearing seams ahead of the excavation was made at Shaft 4 of the Rondout Siphon. This difficult piece of work is fully described by John P. Hogan, Assoc. M. Am. Soc. C. E., in his paper entitled "Sinking a Wet Shaft".*

This shaft passes through water-bearing strata, 131 ft. thick. Nipples were swedged into drill holes, valves put on, and the drilling was continued through the pipe and valve into the softer water-bearing strata, the holes furnishing channels for grouting the rock seams. The method was quite similar to the procedure in the Astoria Tunnel. It was not found necessary to place a concrete blanket in the bottom of the shaft as a bulkhead against which to grout, though, at one time, the rock was so soft that considerable grout was wasted in clogging the surface seams. Altogether, 2 960 bags of cement were injected, and grout tank pressures up to 275 lb. per sq. in. were used, giving a net pressure over and above the ground-water of from 100 to 180 lb. per sq. in. Large seams were especially well filled, the largest one being 8 in. wide, and quantities of water, up to 400 gal. per min., were stopped at different groutings. As excavation progressed, however, there was considerable seepage from the smaller seams; this was not cut off, and, for the entire depth of the shaft, amounted to about 450 gal. per min.

At many other places along the line of the Aqueduct, the leakage from clean-cut open seams was stopped entirely. Near the east shaft of the Hudson River Siphon, an open seam discharging about 500 gal. per min. was completely dried by placing a concrete bulkhead across the heading and forcing in neat cement grout under a pressure of 700 lb. per sq. in., which was 200 lb. greater than the ground-water pressure.

It was common practice in permanent shafts, which were concrete-lined, to place this lining—at least, across water-bearing strata—during the process of shaft-sinking, putting in weep-pipes to vent the water, which pipes were grouted after the concrete had hardened. This method proved effectual in shutting off leaks of considerable size; for instance, a leak of 125 gal. per min. in Shaft 5 of the Rondout Siphon was completely shut off in this manner.

The grouting of the pressure tunnels proper involved one consideration which was not found in the case of the shafts, or in the case of any tunnel which, during use, is subject to normal air pressure. The lining in such a shaft or tunnel is in compression from the pressure

* *Transactions*, Am. Soc. C. E., Vol. LXXIII, p. 398.

Mr. Freeman.

of the surrounding rock, earth, or ground-water. The pressure tunnels of the Catskill System pass deep under the valleys, far below the hydraulic gradient, and will be operated under an outward pressure of from 150 to 650 lb. per sq. in.; although the ground-water pressure relieves much of this, there remain net bursting pressures up to 200 lb. per sq. in. under operation. Consequently, the total net outward pressure is great in the case of tunnels ranging from 11 to 14½ ft. in finished diameter. The concrete lining of such tunnels is not thick enough to hold under tension, and depends on the rock backing to carry the load. Seamy, water-carrying rock furnishes a ready channel to take away water which may escape through the concrete lining. Furthermore, rock which is very seamy and broken does not furnish such a good backing for the concrete lining. Thus, the grout which was injected into the surrounding rock seams, helped in two ways: in solidifying the rock and making a better backing for the concrete, and in closing some channels which would carry away any water that might leak through the concrete lining. To meet this condition, an attempt was made, by using the tunnel lining as a bulkhead, to grout farther back into the seams, and to fill smaller seams than had been done during the process of excavation.

The first trial was made at Shaft 4, north, of the Rondout Siphon, where the tunnel passes through the water-bearing High Falls shale and Binnewater sandstone, both of which are broken, porous rocks, particularly so at this point, as they are twisted and crushed in a fault zone. The length of tunnel in this rock is about 250 ft. During excavation an effort was made to grout off the leakage in the heading, and a little was stopped, but the attempt was not wholly successful. No concrete bulkhead was built against which to grout, and the rock was so weak that the face could not be solidified sufficiently to form a backing for grouting pressures. Finally, a test-hole was driven with a diamond drill 250 ft. ahead through the water-bearing strata. Gradually, the leakage decreased, indicating a lowering of the ground-water level, more pumps were put in, and excavation was completed without additional grouting. The maximum leakage in this section was 2 000 gal. per min. A complete account of this work has been given by B. H. Wait, Assoc. M. Am. Soc. C. E.*

The thickness of the concrete lining through this wet section was 24 in., twice the ordinary minimum thickness. As a drip-pan to protect the soft concrete from the incoming water, and as a form of reinforcement for the tunnel lining, a steel shell was erected for a length of 174 ft. This consisted of 6-in. I-beam ribs bent nearly to a circle, but open at one side, the open ends fitting to the concrete invert which was placed first. For a covering, $\frac{3}{16}$-in. plates were

* *Engineering Record*, Vol. LXIII, p. 660, June 17th, 1911.

Mr. Freeman.

bolted to the ribs, and no attempt was made to secure water-tight joints. The concrete invert contained a longitudinal drain with Y-branches to each side, which picked up the leakage. The tunnel at this point was on a 15% incline, and drained itself readily.

The space between the steel shell and the rock was very carefully dry-packed, and afterward the concrete lining was placed inside this shell. Two sets of grout pipes were used, one leading through the concrete to this dry-packed section, the other set being carried from 1 to 3 ft. into the rock seams.

The dry packing outside the concrete was grouted first, while the pipes to the seams remained open to discharge the rock leakage, except those which were closed temporarily on account of leaking grout, when some of the thick grout worked back into the larger seams. Before grouting the dry packing, the inward leakage was 900 gal. per min., but after this grouting was completed, the deep-seated pipes discharged only 540 gal. per min., as the thick grout had worked back into some of the larger seams and cut down the leakage.

A start was made to grout the rock seams proper by injecting an ordinary mixture (95 lb. of cement to 5½ gal. of water) of neat grout into a deep-seated pipe leaking about 4 gal. per min. and 3 bags of cement plugged the seam. Then, at the suggestion of T. H. Wiggin, M. Am. Soc. C. E., Designing Engineer of the Pressure Tunnels, this pipe was re-opened, cleaned out, and a very thin mixture used (23¾ lb. of cement to 28 gal. of water). Of the latter mixture, 24 bags were injected into this same hole which had refused at 3 bags of the thicker mixture. The use of the very thin grout was continued with grouting pressures up to 300 lb. per sq. in. The tunnel-service pressure was 100 lb. per sq. in., higher pressures being furnished by a Westinghouse (booster) compressor. Two compressors were piped with two grout tanks, so that either compressor could be used with either tank. With one tank charging and the other discharging, the stream of grout was kept practically continuous. The rock-water pressure grouted against was 20 lb. per sq. in. If a pipe took grout easily at a pressure of 100 lb., the mixture was thickened a little (47½ lb. of cement to 28 gal. of water), and, if this continued for some time, the thickening was increased to 1 bag per 28 gal. of water. When the injection pressure began to rise, the mixture was thinned again. Pipes were considered full which refused the thinnest grout under a pressure of 300 lb. It was noticeable that the grout traveled some distance (from 30 to 40 ft.) in the seamy rock, showing up at other pipes, and cutting down the leakage at those pipes and filling the seams back of them. Not more than one in ten of the pipes leading to the rock seams took any considerable quantity of grout, though every pipe was tried. After all grouting was completed in this section, the inward leakage was

Mr. Freeman.

less than 1 gal. per min., due, perhaps, to a good concrete lining and not wholly to grouting. In a length of 174 ft. of tunnel, 940 bags of cement were injected into the rock seams, the finished diameter of the tunnel at this place being 14½ ft.

After the ground-water level was restored, four test holes were drilled about 3 ft. out into the grouted rock, where heavy leakages had occurred at the time of excavation. Two of these holes were practically dry, the third discharged ½ gal. per min., but the fourth struck a water-bearing seam and discharged 71 gal. per min. The latter hole was in the bottom of the tunnel, the others were at the horizontal diameter and the top. It is conceivable that the bottom might show the poorest results, as the ultimate direction of grout flow is upward. A gauge reading showed the rock water pressure to be 117 lb. per sq. in.

Another section of the same siphon, through blocky, seamy Helderberg limestone, took 946 bags of thin grout in a length of 50 ft. Two test holes, 3 ft. into the rock, showed no leakage.

Though absolute assurance cannot be given that all the seams were filled, the volume of cement injected indicates that many of them were closed by this method, which thicker grout would have clogged at the openings and left unfilled.

A difficulty arose in grouting a combination of large and small seams, as the large seams furnished an easy egress for thin grout, and the small seams clogged at the entrance with the thicker mixture. By increasing the cement per batch, if the pressure of injection remained low, and decreasing it again as the pressure rose, an attempt was made to adjust the consistency of the batch to the size of the seam.

Mr. Bradley.

W. H. Bradley, Esq.*—At a time when the outlook on the tunnel work was very dark, a prominent official of the Company said to the speaker "I was informed by an engineer that the tunnel is a failure—that it will never be completed". The speaker replied "You have never heard me say that, when you do you may take it to be a fact; at present it is not a fact".

To carry Mr. Davies' description of the pipe testing a little farther: About 2 weeks ago, one of the main pipes in the tunnel was tested, first by filling it with water which was allowed to rise about 40 ft. in the stand-pipes, giving a pressure of about 20 lb.; the joints were then inspected thoroughly for possible leaks, after which the water was run out and an air pressure of 40 lb. per sq. in. was applied, each joint being inspected by applying soap and water. After this inspection, water was again put in, the stand-pipes being filled to the top of the shaft, which created a pressure of about 125 lb. per sq. in. in

* Chief Engineer, Consolidated Gas Company, New York City.

Mr. Bradley.

the pipe. This may seem to have been unnecessary—after the previous tests—but it was a precautionary measure intended to expel all air from the pipe, because air and gas form an explosive mixture and there is likely to be serious trouble if a light reaches it. When the pipe was filled with water, the gas valves at both ends were opened, and, as the water was drawn out, the main was filled with gas with no mixture of air. This main is now in daily use.

The same procedure is being followed with the second main.

The head-houses are now being erected over the tunnel shafts, the elevators, pumping plant, telephone and lighting circuits are being installed, and the work will soon be completed.

Mr. Sanborn.

JAMES F. SANBORN,* ASSOC. M. AM. SOC. C. E.—It would be interesting to have a description of the difficulties in putting in the vertical pipes in the shaft. Were any curving rings or shims required? The speaker asks for this information because, in the work of the New York Board of Water Supply, there was trouble in erecting the risers of the City tunnel shafts, which were steel pipes, and it was found expedient to use a hub and spigot arrangement.

Mr. Wegmann.

EDWARD WEGMANN,* M. AM. SOC. C. E.—This paper is a very valuable contribution to the engineering literature of tunneling, and, doubtless, will be consulted by other engineers who encounter difficulties similar to those met in the Astoria Tunnel. The paper is very complete, as it describes in detail, not only the construction of the tunnel, but, also, the plant and methods used.

The speaker is particularly interested in the paper because he was one of the engineers on the tunnel, constructed in 1885 to 1891 under the Harlem River as part of the New Croton Aqueduct, in driving which decomposed rock, similar to that described by the author, had been encountered. The tunnel was about 1 300 ft. long and required a circular excavation, 14 ft. in diameter.

Before the work was begun, numerous diamond-drill borings were made to ascertain the character of the rock through which the tunnel was to be driven. Near Manhattan Island the rock was found to be gneiss, but for the greater part it was limestone having many seams. A pocket of decomposed rock was discovered in the gneiss formation, and was investigated by making borings about 50 ft. apart.

Based on the indications of the diamond-drill borings, the tunnel was planned so as to have its invert about 155 ft. below mean tide. It was thought that at this level there would be at least 30 ft. of solid rock above the tunnel, but when the heading had advanced about 300 ft. from Manhattan Island, a seam of rock, full of water, was struck by the drills. Water poured into the tunnel, and the workmen ran

* New York City.

for their lives. Sand and stone soon filled the drill holes, and made it possible to close them with wooden plugs. Mr. Wegmann.

A diamond drill was taken into the heading, and the ground in front and below the heading was explored by borings. In order to insure safety while this work was going on, a strong timber bulkhead, with a suitable gate, was built in the heading, just back of the diamond drill.

The test borings showed that the seam of decomposed rock extended about 75 ft. below the tunnel, instead of terminating 30 ft. above it, as had been believed by the engineer. In front of the heading, the seam was found to be 28 ft. wide, and beyond it there was limestone containing seams in which the water was under the full pressure due to the river, *i. e.*, under a head of more than 150 ft.

The important question arose as to how the tunnel was to be constructed through the seam of decomposed rock and through the limestone. Negotiations were held with the company controlling the patents for the freezing process, which had been used very successfully for sinking shafts through quicksand. This company was very willing to undertake the work on a percentage basis, but would not give any guaranties with regard to success.

The engineers in charge of the work were anxious to try to drive a pilot tunnel through the soft ground, but the contractor who was to do this work was very loath to undertake it, as it necessitated buying large pumps. Apparently, he had more influence with the Aqueduct Commissioners, under whose direction the New Croton Aqueduct was built, than the engineers of the Commission. By an order of this Commission, the engineers were directed to abandon the 300 ft. of tunnel that had been excavated to the soft seam, and to start another heading about 307 ft. below mean tide. At this level no difficulties were encountered.

The only additional cost of constructing the tunnel at the new level was in the lowering and hoisting of materials, but this was more than offset by the fact that the tunnel was very dry at the lower level. In spite of these circumstances, the contractor, assisted by able counsel, sued the City of New York for seventy items, aggregating about $350 000, for the additional cost of constructing the tunnel at the lower level, and for some other extras. The original contract price for the work was only about $500 000.

The lawsuit against the City was begun in 1890, but the contractor's lawyers waited 16 years, before they brought the case to trial. They thought, probably, that their chances of success would be better when the engineers and inspectors, who had been connected with the work, had gone West or died. The City won the suit twice, but the Court of Appeals sent it back a third time for re-trial. The case was about finished before a referee, when this gentleman, un-

Mr. Wegmann. fortunately, died. Twenty-four years after the suit was begun, it was settled out of Court by the City paying the contractor $25 000 on claims amounting to $350 000. It would be interesting to know what part of the $25 000 was received by the contractor's lawyers and how much by the contractor himself. The speaker mentions this lawsuit to show some of the difficulties, other than soft seams, which engineers encounter in tunneling.

Mr. Kellogg. R. C. KELLOGG,* ASSOC. M. AM. SOC. C. E.—The Astoria Tunnel is one of the principal connecting links in a vast scheme of gas distribution which comprehends sending out from the Astoria works, at some future time, approximately 200 000 000 cu. ft. of gas every 24 hours, part of which may eventually be pumped as far as Albany. This will not only rid Manhattan Island of unsightly gas manufacturing structures, but will also benefit the towns through which the supply may travel. The Astoria gas plant itself, when completed according to the plans conceived, will contain ten units, each capable of manufacturing 20 000 000 cu. ft. of gas every 24 hours, and ten storage holders each having a capacity of 15 000 000 cu. ft.

Mr. Brush. W. W. BRUSH,† M. AM. SOC. C. E.—It may be of interest to mention the fact that the engineers of the New York Board of Water Supply studied a line for a water supply tunnel which would follow somewhat closely that of the gas tunnel the construction of which has been described so clearly and interestingly by Mr. Davies. The future water supply of New York City will necessitate the delivery of large quantities into Queens Borough, which contains the maximum area available for development within the city, and where eventually an enormous manufacturing center will probably be found, with a large consumption of water.

The studies made several years ago to locate the tunnel to deliver the Catskill supply to the five boroughs indicated that a rock tunnel could probably be driven successfully into Astoria and carried through the rock into the Borough of Brooklyn, but that the cost would be decidedly greater than a tunnel along the adopted line under the Bronx and Manhattan to Brooklyn. One of the uncertain elements of cost was the driving of the tunnel under the East River, probably passing under Randall's and Ward's Islands. The question of determining the character of the rock at the contacts between the various formations was deemed to be a difficult problem.

Ultimately, however, in all probability, a tunnel will be driven under the East River, near the Gas Company's tunnel, for the purpose of delivering an additional water supply to the Borough of Queens and in part to the Borough of Brooklyn. The experience of the Gas

* Brooklyn, N. Y.
† New York City.

Company will then furnish especially interesting and valuable information for the engineers who will lay out the line and grade of the later tunnel. A water tunnel can readily be driven at any depth desired, and therefore the engineers may avoid some of the difficulties mentioned by Mr. Davies. Mr. Brush.

The riser pipes in the shafts of the Astoria-Bronx tunnel are embedded in concrete, and are therefore self-supporting.

THOMAS H. WIGGIN,* M. AM. SOC. C. E.—In the Catskill Aqueduct pressure tunnel siphon the rock conditions were similar to those described by the author. The first case was in the sinking of Shaft 4 of the Rondout inverted siphon which crosses the valley of Rondout Creek near High Falls, west of Kingston, N. Y. This was described in a paper† by J. P. Hogan, Jun. Am. Soc. C. E., and a discussion by the speaker adds other data, notably some from a very interesting shaft of the Detroit Salt Company. In Shaft 4 of the Rondout Siphon heavily water-bearing rock was encountered, as had been predicted from the preliminary borings and pumping tests in certain of these borings. At a depth of about 225 ft. the inward flow was more than 800 gal. per min. At first ordinary drill holes, and finally diamond-drill borings, made around the periphery of the shaft from the level to which sinking had progressed, were grouted, after which excavation was resumed with continued precautions as to drilling, carrying a pilot hole ahead and grouting through water-bearing drill holes. The leakage was very much reduced by the grouting, and would doubtless otherwise have reached very large quantities. In grouting this shaft, 971 bbl. of cement were used. Some of the seams were wide, 8 in. being the maximum. Mr. Wiggin.

The grout when encountered in the seams during subsequent excavation was like moderately soft limestone, and showed very fair strength.

Later, in the tunnel driven from this shaft, the same wet rock strata were encountered, again in the position expected from the borings, and similar grouting was done in advance of excavation, though not so successfully. The maximum leakage into the tunnel in this stretch was about 2 000 gal. per min. This did not interfere so much with operations as it might have under other conditions, because the tunnel was driven up on a 15% grade. The water made a very pretty cascade. A heavy concrete bulkhead, with a door, had been built between this stretch of tunnel and the shaft, in order to avoid loss of the tunnel in case of meeting a sudden inflow.

Much of the rock in this piece of tunnel was divided into very small blocks by water-bearing seams, and the impression was gained,

* New York City.

† *Transactions*, Vol. LXXIII, 1911, p. 398.

Mr. Wiggin. from such grouting as was done, that general impregnation by grout would be impossible with any practicable number of holes. Hence, the leakage was taken care of by a system of bottom drains and a shield of steel angle ribs and plates laid shingle-wise, which kept the water off the concrete while it was being placed. The space behind the steel was at first "dry-packed" with rock fragments, and was grouted after the concrete lining had been placed within the shield. The drains were also grouted. This method resulted in making this very wet portion of the siphon almost bone dry when completed.

In the Hudson Tunnel, where grouting pressures of about 1 000 lb. per sq. in. were used, the rock is generally solid and hard. It is what most people, from casual examination, would call a granite; geologists call it granodiorite. In general, the rock was rather tight, but within 2 or 3 ft. of a drill hole that was almost dry, another hole struck a flow of about 200 gal. per min. which temporarily overcame the pumping facilities causing the heading, which was then about 275 ft. long, to fill.

After the tunnel was pumped out, the leakage was collected in pipes, and a concrete bulkhead was placed at the end of the heading and anchored to the rock by steel rods. Four or five additional drill holes were made into the water-bearing region, which was identified by feelers as a narrow seam across the tunnel. The combined flow of all holes previous to grouting was found to be about 550 gal. per min., but was under control by gated pipes. A ground-water pressure of 400 lb. per sq. in. was recorded when the pipes were closed. The seam was finally grouted through the drill holes under a pressure of 1 000 lb. per sq. in. The tunnel at that point is more than 1 100 ft. below sea-level, and this high grouting pressure was required to overcome the ground-water head and the friction of the grout.

The equipment generally used on the Catskill works for grouting is a tank with top door and various pipes arranged so that the grout may be mixed by the release of air through the bottom pipe and ejected by forcing air into the top of the tank. This is the so-called Canniff grouting machine. In the Hudson Tunnel, air at a pressure of 1 000 lb. was not available for forcing out the grout. At first the attempt was made to pump the grout by plunger pump, but the pump valves soon wore out. The pump was then repaired and used to force water instead of air into the grouting machine on top of the grout, thus causing gradual dilution as well as displacement of the grout, but doing perfect grouting nevertheless.

A heavy concrete bulkhead, with a thick hinged cast-steel door, had been placed across the tunnel to prevent another flooding, and a liberal pumping equipment was put in. The tunnel was then driven through the wet ground, and a narrow seam which had been grouted could be

seen across the roof and sides of the tunnel, with only a few drops of water coming from it here and there. Mr. Wiggin.

This Hudson Tunnel, which is about 3 020 ft. long, had been explored by about a dozen vertical holes and by two pairs of inclined borings, the longest of which was about 2 052 ft. The holes of each pair of inclined borings passed each other near the middle of the river and solid core was taken from all the holes, so that it was known that the tunnel would be in good granite nowhere less than 150 ft. thick above the tunnel. Although considerable water in the aggregate was encountered in these holes, as is usual in granite, so large a flow concentrated at one seam was surprising, particularly as at that point the rock was not less than 700 ft. thick above the tunnel. This experience serves to emphasize the fact that in many kinds of rock very wet seams may be found, surprisingly close to very dry ground and under a thick roof of apparently sound rock.

The speaker has been very much interested in Mr. Davies' paper, and particularly in the success of the method of grouting a large number of holes, and grouting back into the body of the rock. This process has been demonstrated very beautifully by this tunnel, and also by the Catskill work, and is doubtless bound to be used more and more in difficult ground. At the same time, the speaker thinks that engineers who have had experience with wet ground will be inclined more and more to keep away from it, even at a considerable expense for exploration and the deepening of the tunnel.

On the Catskill work it was found that depth was no particular drawback. When the engineers started planning pressure tunnels, 200 ft. was thought to be pretty deep for such purposes. Some of these early studies look rather absurd in the light of the finished work. The idea of liberal depth gradually gained force; and if some of the work were to be done over again, even greater depths would probably be used in places.

WILLIAM CULLEN MORRIS,* M. AM. SOC. C. E. (by letter).—In relation to this paper, some information relative to the accident and casualty features of the work may be of interest. At the start of operations the Company decided, in view of the high rates prevailing for work of this class, to undertake the insurance, instead of placing it with a casualty company. A very liberal policy was adopted by the Company in the treatment of the men, allowing them full time during incapacity owing to any cause (either illness or accident) and furnishing medical attendance and supplies during the period of disability. Field hospitals were maintained at each end of the tunnel, with orderlies in constant attendance. Arrangements were made with neighboring hospitals for prompt ambulance service in case of emergency, and every Mr. Morris.

* New York City.

Mr. Morris. precaution was taken so that injuries could be treated to the best advantage. A physician was employed to supervise each of the hospitals, and the orderlies reported directly to these physicians all cases of first aid or disability treatment.

A force was organized for continuous inspection of the work, with a view of eradicating all preventable causes of accidents, the inspectors patrolling the work and immediately bringing to the attention of the tunnel force any features they might notice which might result in accidental injury. This feature of the work created some confusion for a time, but after several weeks of trying out, the arrangements were perfected to such an extent that there was no interference with rapid progress.

The accident treatment record and accident prevention were all under the direct supervision of the Company's Engineer, Mr. Harold Carpenter, who reported to the Company's construction office.

The record of accidents throughout the work is as follows:

Total accidents	1 525
Serious accidents	115
Total time lost	1.78%
Cases sent to outside hospitals	24
Cases treated in tunnel hospital	67
Cost of accidents in relation to pay-roll	5.36%

Mr. Davies. JOHN VIPOND DAVIES,* M. AM. SOC. C. E. (by letter).—The very interesting contributions by Messrs. Freeman, Wiggin, and Wegmann, as to the experience in the Catskill Aqueduct, illustrate the principal points of difference between that undertaking and the Astoria Tunnel. In the latter the primary feature was the consolidation of the soft and partly decomposed rock in order to permit of the construction of the tunnel, and, secondarily, the filling of the fissures, for the purpose of stopping leaks. The primary feature in the case of the Catskill Aqueduct was the absolute filling of all voids and the stoppage of leaks, and, secondarily, the consolidation of the rock.

For the stoppage of leaks and the consolidation of the rock, considerable grouting had been done in 1892 and 1893 in the construction of the East River Gas Company's tunnel at 70th Street, Manhattan, referred to in the paper. At that time the writer ordered cement, specially burned, with the elimination of any plaster admixture, so as to obtain the most rapid setting material. That tunnel was constructed at a comparatively shallow depth, and there was so little cover between it and the bed of the river that high pressure could not be used. The same condition occurred in the construction of the Hudson and Manhattan Tunnels, where large quantities of cement grout were also

* New York City.

used for the purposes just referred to, and, in that case, also, the comparatively shallow depth of these tunnels precluded any high-pressure grouting. In this latter case it was found that grout would follow the line of least resistance and pass out with little interruption into sewers, drains, or watercourses, without generally spreading into the soil, to give the results which were attained with the high pressures and greater resistances encountered in the Catskill Aqueduct and the Astoria Tunnel. Mr. Davies.

Mr. Sanborn inquires as to the erection of the vertical risers. These pipes were of cast iron, $2\frac{3}{4}$ in. thick, with hub and spigot joints. It was specified that they were to be machine-faced on the bearing surfaces in hubs and ends of spigots, so that there should be a true fit at every joint. Unfortunately, the facing of the two ends was not executed from common centers, the result being that these pipes—which as designed should have given perfect vertical alignment—were, when erected, found to be in error, with a regular creep from the vertical, due to imperfections in foundry work, and this necessitated the use of wrought-iron shims to bring them into correct vertical alignment.

Mr. Lavis refers to the relations of mining engineers to the various tunnel operations now being carried on in New York City. It occurs to the writer that there is a general misunderstanding as to the use of the term "mining engineer" in this respect. In common with other engineers engaged on subway construction in New York City, the writer has been, to a considerable extent, brought up with coal mining and other underground operations. The timbering and methods of construction used by coal and metalliferous miners are as far removed from the methods of the engineer engaged in subway work as it is possible to conceive. The class of mining involved in subway and tunnel construction is distinctly a development of business, entirely apart from the ordinary practice of those engineers engaged in such purely mining operations. The men employed in England are known as "miners", but this class consists of those essentially engaged on engineering work in heavy soft ground tunneling, not in mining operations. A large majority of those engaged in this business in New York have learned their trade in Europe, or from those who have brought the art from Europe, and there should be no conflict in the distinction between so-called mining engineering and engineering connected with tunnel work. The so-called mining engineer knows little of the type of timbering used in these tunnels.

Mr. Morris calls attention to the provisions for attending to accidents and casualties. The writer desires to state that in the treatment of employees injured or incapacitated on this work, the Astoria Light, Heat and Power Company provided more liberally than any

Mr. Davies.

employer with whom he has had any previous experience. As there were no hospitals in the vicinity of the shafts, the officers of this company decided to build them especially for this work; they equipped them completely, not only for rendering first aid, but also for the actual treatment of cases under competent medical supervision. The adjustments with employees were most liberal, and it is of interest to note that, notwithstanding this great care and the liberality in all cases, the entire cost of the accident account amounted to only 5.36% of the pay-rolls.

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Paper No. 1360

CONCRETE-LINED OIL-STORAGE RESERVOIRS IN CALIFORNIA: CONSTRUCTION METHODS AND COST DATA*

By E. D. Cole, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. C. P. Bowie, Emile Low, Ralph J. Reed, and E. D. Cole.

Synopsis.

This paper describes briefly the construction of concrete-lined earthen reservoirs used for storing crude petroleum in the oil fields of California, giving in detail the methods used to obtain safe and economical results, together with the total cost of storage per barrel of capacity.

The method used in building up a compact and unyielding embankment as a backing for the concrete lining is given special attention.

The losses of oil, due to evaporation and seepage, are given for both lined and unlined reservoirs, showing the economy of the concrete lining.

Discussion is requested in particular on the question of expansion joints in work of this class, and also on the permeability of concrete subjected to the pressure of crude oil at low heads.

Quite a number of large earthen oil-storage reservoirs have been constructed in the numerous oil fields of California, and the

* Presented at the meeting of September 15th, 1915.

writer, being unable to find any literature covering this subject, and having been actively engaged for the past 3 years on construction of this class, thought that a brief description of the methods used and some unit costs would be of interest to engineers.

In general, these reservoirs are circular in plan, with earthen embankments built up from material excavated from within the reservoir, and range in capacity from 500 000 to 1 000 000 bbl. Some of the larger ones are oblong in plan, with circular ends connected by a rectangular section in the middle. In many cases dependence has been placed on the earth alone to prevent loss of oil by seepage, but it is now becoming general practice to line the inside slopes and bottom with a thin layer of reinforced concrete as an additional safeguard against leakage. The reservoirs are covered with a wooden roof supported usually by 6 by 6 or 8 by 8-in. posts resting on wooden footings or concrete piers. Two of the largest of these reservoirs were built entirely of reinforced concrete by the Union Oil Company at San Luis Obispo, Cal., in 1906, a partial failure of one of which has been noted in the technical press.*

It is the purpose of this paper to describe the construction of one of two 750 000-bbl. reinforced concrete-lined reservoirs, built by the writer, for the Kern Trading and Oil Company, at Bakersfield, Cal., during the winter and spring of 1913-14. The general dimensions of the reservoirs are as follows:

Inside diameter (bottom)	462 ft.
Inside diameter (top)	528 "
Depth	22 "
Width of top of embankment	11 "
Inside slope	1½ : 1
Outside slope	1½ : 1
Thickness of concrete lining (bottom)	3 in.
Thickness of concrete lining (top)	2½ "

Earthwork.—The formation at the site was a light sandy clay, and this was easily handled by the Fresno and wheel scrapers used throughout the work. After the site was cleared of all brush and grass, the foundation under the embankment was thoroughly plowed and wet down before the fill was started. Water for moistening the material

* *Engineering News*, March 16th, 1911, p. 310.

was supplied through a 2-in. pipe line laid around the site, just outside the outer line of slope stakes, with hose connections approximately 100 ft. apart. A 2-in. line was also run to the center of the reservoir to supply water to the portion of the site which could not be reached from the outside line. To avoid being in the way of the scraper teams, this pipe line was laid through one of the three 12-in. outlet pipes that were placed in position under the fill at the beginning of the work. A narrow trench was dug from the inner end of this outlet pipe to the center of the reservoir, and the 2-in. line was laid in it. This line was lowered from time to time as the work progressed, and was kept far enough below the surface of the excavation to be clear of the plow and scraper teams. Wetting down the excavation material was a help in several ways, as it not only made a more compact bank, but kept the dust down and made the earth ride better in the scrapers. This may seem inconsistent, inasmuch as the work was done during the rainy season, but can be readily understood on taking into consideration the fact that only 6 working days were lost during the winter on account of wet weather. The embankment was built up in thin layers, about 3 in. thick, laid parallel to the floor of the reservoir, and well compacted. In addition to the tramping of the scraper teams, the fill was compacted by two petrolithic road tampers which were driven continually around the top of the embankment.

To insure a compact and uniform backing for the concrete lining, on the inner slope below the natural ground surface, the excavation was started 2 ft. (measured normal to the slope) inside the inner line of slope stakes. This necessarily increased the quantity of excavation, and left the embankment short on the inner slope by this quantity. After the completion of the main portion of the embankment, a lining of selected material, 3 ft. thick (measured normal to the slope), was built up against the inner slope, from sub-grade (1 ft. below floor grade) to the top of the finished fill. This extra foot of material was put on to insure a compact surface at the grade line of the inner slope, but was afterward removed, as will be described later. The refill is an important part of the construction because it would cut off any layers of sand or loose material that might be encountered in that portion of the inner slope which lies below the ground surface. On some previous work it was found necessary to excavate and refill portions of the natural embankment, below the ground surface, after

the fill had been completed and trimmed to grade; this work caused considerable delay and added expense. One concrete-lined reservoir in this field partly failed, due to neglect of this part of the work, necessitating heavy expense in emptying the reservoir, besides the loss of a considerable quantity of oil and the cost of patching the lining.

Fig. 1 illustrates the method used on the first reservoir, in which a refill was put in. This particular section of the embankment was all in cut, in a loose gravel bank, making it imperative to put in a lining of sandy clay as a backing for the concrete. This refill or lining of selected material was started with Fresno scrapers, but these were soon abandoned in favor of wheel scrapers, on account of the difficulty in keeping them from sliding over the edge, and also because the wheel scrapers would build up a bench of the required width, and it was impossible to keep within the limits with the Fresnos. Apparently, the wheel scrapers build up a more compact lining, as will be noticed by referring to Fig. 2, which gives a close view of this particular piece of bank after it had been completed and trimmed off, and had been exposed to a heavy thunder shower. The lower portion of this view shows the upper edge of the refill which was put in with the Fresnos, and the upper portion shows the work done with wheel scrapers.

Trimming Slopes.—On the completion of the main embankment and the refill, the excess material on the inner slope, which ranged in thickness from 1 ft. at the top to 2 ft. at the bottom, was trimmed off leaving the slope smooth and true to grade. For this work of trimming the slope, a novel and (the writer believes) original method was used. Grade stakes were set on radial lines, both at the top and inner toe of the slope, approximately every 10 ft. around the circumference of the reservoir. Men with mattocks and slope-level boards then dug narrow trenches, 1 ft. wide and true to grade, from the top grade stake to the stake at the toe of the slope. Then 2 by 4-in. timbers, 38 ft. long, each faced with a narrow strip of strap iron, were placed in the bottom of each trench to act as guides for a trimming machine which was used to finish that portion of the slope between the hand-dug trenches. Before using the planer, however, all excess material above the top of the 2 by 4-in. timbers was scraped off the slope with a specially made Mormon or Buck scraper which was dragged up and down the slope, power being furnished by a double-drum hoisting

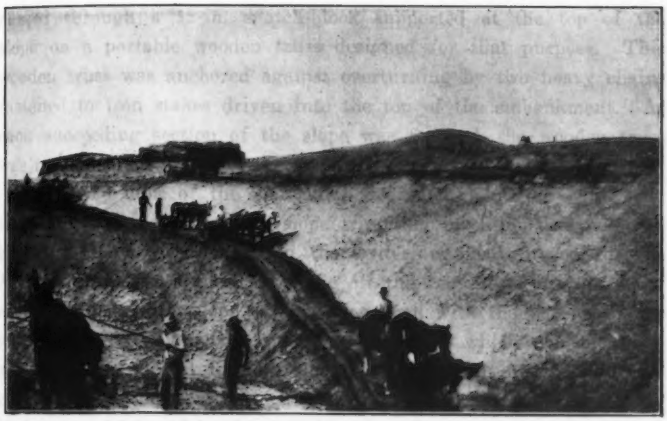


FIG. 1.—METHOD OF PUTTING IN REFILL OR LINING OF EMBANKMENT OF OIL-STORAGE RESERVOIR AS A BACKING FOR CONCRETE.

FIG. 2.—REFILL OR LINING OF EMBANKMENT OF OIL-STORAGE RESERVOIR COMPLETED AND TRIMMED OFF. UPPER PORTION PUT IN WITH WHEEL SCRAPERS, AND LOWER PORTION WITH FRESNOS.

engine at the center of the reservoir. The back-up line from the engine passed through a 12-in. snatch-block supported at the top of the slope on a portable wooden truss designed for that purpose. This wooden truss was anchored against overturning by two heavy chains fastened to iron stakes driven into the top of the embankment. As each succeeding section of the slope was finished, the wooden truss was moved along the top of the bank with a team of horses.

After the bulk of the material above the top of the 2 by 4-in. timbers had been removed, a slope trimming machine, designed and built by Mr. C. O. Zeller and the writer, was substituted for the Mormon scraper and used to plane off the remaining thin layer of earth and bring the slope to grade, or flush with the bottom of the guides. Fig. 3 shows the trimming machine, which consists of a rectangular frame, 11 ft. long and 6 ft. wide, built up of 6-in. steel channels bolted together and carrying two cutting blades. The cutting blades are of $\frac{3}{8}$ by 12-in. flat rolled steel, and are set at an angle with the frame of 1 to $2\frac{1}{2}$. The blades are also set at a slight angle longitudinally with each other, and the cutting edge projects down 2 in. below the bottom of the frame. The planer is dragged back and forth on the slope until the ends of the frame ride on the top of the guides, and that particular section is shaved off flush with the bottom of the guides, or down to grade. In this way nearly nine-tenths of the slope were finished by machine and at one-half the cost of doing the work by hand. One might naturally suppose that there would be considerable difficulty in keeping the guides in place without staking them down, but very little trouble was experienced. Loose earth would fill in the trenches around the guides as the machine worked above them, and this served to hold them in place. The trenches being dug on radial lines necessarily made them closer together at the bottom of the slope than at the top, but the blades of the planer were long enough to catch all the earth between trenches at the top of the slope and still have room at the bottom without binding between the guides. Fig. 4 shows the Mormon scraper working on a $1\frac{1}{2}$ to 1 slope in light sandy soil, and Fig. 5 on a 1 to 1 slope in a very heavy clay soil. An idea of the compactness of the face of the embankment can be obtained by a close inspection of Fig. 5, in which are to be seen the marks left by the teeth which it was found necessary to put on the blade of the Mormon to make it dig into the embankment. Fig. 6 shows

the planer working on a 1½ to 1 slope, and Fig. 7 on a 1 to 1 slope. The crew on all the trimming work consisted of only seven men, including the engineer who ran the hoist.

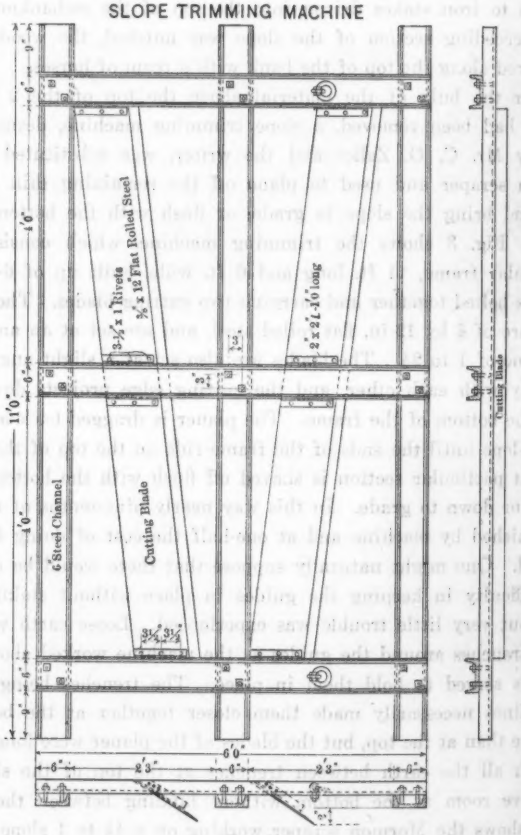


FIG. 3.

Floor.—The floor of the reservoir was excavated to a depth of 1 ft. below grade, and the sub-grade was plowed and well rolled before commencing the refill upon it. It was then back-filled to grade, built up in thin layers, moistened, and well rolled with petrolithic road tampers.

FIG. 4.—MORMON SCRAPER WORKING ON A 1½:1 SLOPE IN LIGHT SANDY SOIL.

FIG. 5.—MORMON SCRAPER WORKING ON A 1:1 SLOPE IN HEAVY CLAY SOIL.

FIG. 6.—SLOPE TRIMMING MACHINE WORKING ON A $1\frac{1}{2}$:1 SLOPE.

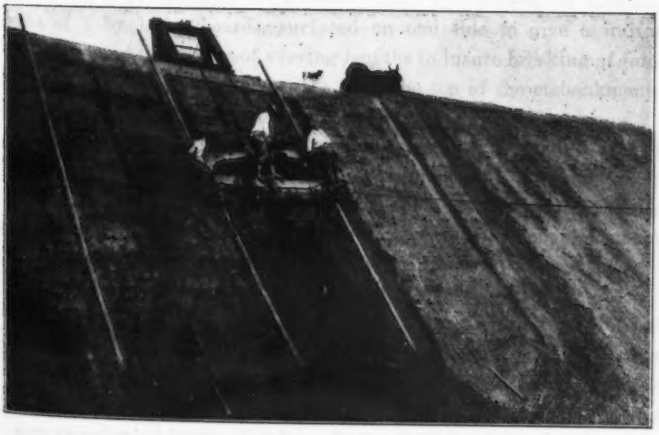


FIG. 7.—SLOPE TRIMMING MACHINE WORKING ON A 1:1 SLOPE.

Sand and gravel patches in the floor below sub-grade were excavated to a depth of 3 ft. below floor grade and then refilled with selected material. The floor was rolled smooth and hard to grade with a 5-ton road roller, a variation of only 0.05 ft. from true grade being allowed.

Pier Footings.—Excavations for the pier footings were made and the concrete piers for the roof supports were poured. Before pouring the concrete piers, however, and after the footings were dug, the floor reinforcing metal, which consisted of one layer of Clinton electrically welded fabric, made up of No. 6 wire in 6-in. square mesh, was laid out over the floor and tied together. The piers were poured through this metal, and later, when the floor was poured, this method left the reinforcing metal continuous throughout the floor and piers, without any patchwork.

Roof.—The roof was supported on 6 by 6-in. posts, approximately 18 ft. apart on centers, resting on concrete piers, and placed in concentric rings around the floor of the reservoir. The rafters were of 2 by 8-in. lumber, 20 ft. long, and were supported by 3 by 12-in. girders, 18 ft. long, which were notched into the tops of the posts. The rafters were laid on radial lines toward the center post, and 2 ft. from center to center at their inner ends. The roofing, or sheathing, was of 1 by 12-in. boards, surfaced on one side to give a uniform thickness, and these were of varying lengths to insure breaking of joints. The rafters that bridged the space from the top of the embankment to the first row of posts at the toe of the slope (see Fig. 12), were of 2 by 14-in. lumber, 38 ft. long, and rested at their outer ends on 2 by 12-in. redwood mudsills on the top of the embankment. All posts and girders were cut to length before commencing the construction of the roof. Later, this lumber was hauled to the top of the embankment and skidded down to the bottom of the reservoir where it was loaded on hand-trucks and distributed.

Bents, consisting of two posts and a girder, were framed in the bottom and hoisted into position with a small hand derrick operating on the roof. This derrick consisted of a boom made up of two pieces of 2 by 8-in. lumber, 38 ft. long, supported in the middle on an ordinary two-wheeled lumber truck, and rigged with a 2-ton hand-operated hoisting crab. The hoisting line ran over a small sheave at the outer end of the boom, and consisted of $\frac{3}{4}$-in. pliable steel cable. While raising a bent, the outer end of the boom was anchored to a rafter

with a U-shaped bolt which could be quickly attached. Later, this system was changed somewhat, as the strain on the girder under the wheels of the derrick was considered too great for safety. The hoisting winch was transferred to the extreme end of the boom, which placed the men operating it over the second row of posts back from the edge of the roof, and only one post and a girder were raised at a time (Fig. 8). This method worked very well, and good time was made. The best day's work consisted in raising 145 bents in 9 hours, or about one-fifth of the total number in the reservoir. In fact, good time was made throughout the job, as the roof, consisting of about 600 000 ft., b. m., was erected in 7 days, with an average of 50 men.

In hoisting the bents, the derrick worked on only one row at a time, starting at the outer row and working toward the center of the reservoir. The hoisting crew was followed by a crew which placed the rafters, and this one was followed by a crew laying the sheathing. Joints in the sheathing were made on top of the rafters, and, on account of the large quantity of sawing necessary, a special rig was devised for this purpose. A 2-h.p. electric motor, belt-connected to a circular saw, Fig. 9, was mounted on a small hand-truck which could be wheeled around on the roof, and used to cut off the ends of the sheathing. Pencil marks were made where boards were to be sawed, and this work was done by 3 men, including the saw filer. Power was supplied by a near-by power line, the transformers being mounted on a pole just outside the embankment. Wires were run to the center post, and lead lines from there to the saw could be dragged to any part of the roof.

Concrete Lining.—The floor and slopes of the reservoir were covered with a lining of concrete reinforced with a continuous layer of Clinton electrically welded fabric, made up of No. 6 wire in 6-in. square mesh. The thickness of the floor lining was 3 in., and that of the slope 2½ in. The concrete was mixed in a Ransome batch mixer, of 1 cu. yd. capacity, placed at the outside toe of the slope, and was delivered into a hopper at the toe of the slope inside the reservoir by a hoisting tower and a 1-cu. yd. automatic dump bucket. From the hopper it was delivered into concrete buggies and wheeled to the spreading gang. In pouring the floor, a 1½-in. layer of concrete was first spread over a small section, and the reinforcing metal was then raised up through the concrete before the second layer was poured. After

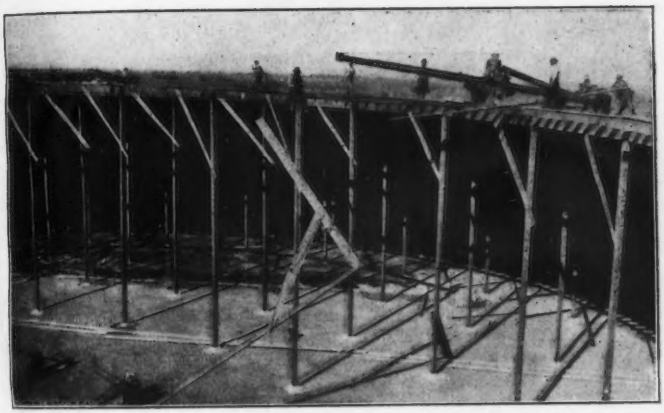


FIG. 8.—METHOD OF HOISTING BENTS INTO POSITION.

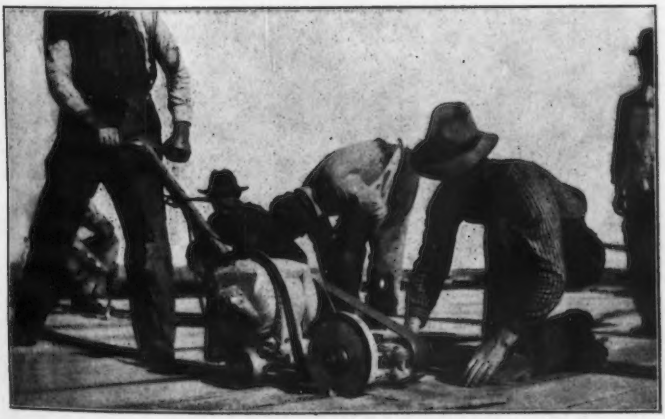


FIG. 9.—ELECTRIC MOTOR CONNECTED TO CIRCULAR SAW, USED TO CUT OFF ENDS OF SHEATHING ON ROOF OF OIL-STORAGE RESERVOIR.

pouring the slab, the concrete was tamped with wide-faced wooden tampers, and troweled smooth and even. The construction joint at the end of a day's work was beveled off, and, before placing concrete against it, on the following day, was swept clean of loose material and thoroughly grouted with a mortar of neat cement.

The concrete for the slope was delivered from the hoisting tower into small hoppers set on wagons and hauled with teams around the top of the embankment. From the wagons it was poured directly on the slope; no forms of any kind were used, as the concrete was mixed so that it would not run any more than enough to facilitate spreading. The slope lining was also put on in two layers, in order to insure that the reinforcing metal would be placed in the middle of the slab. No trouble was experienced in pouring the slope without the use of forms, as the concrete was mixed fairly "stiff", and was prevented from "sluffing" by the reinforcing metal. In fact, the writer has just finished lining a reservoir with side slopes of 1 to 1, for the Union Oil Company, at San Luis Obispo, Cal., without the use of forms and with no appreciable difficulty. Each day's work was gone over on the day following and swept with a cement wash mixed in the proportion of 1 of part cement to 1½ parts of water. This wash of grout was applied to fill up any small checks which might appear while the concrete was taking its final set.

No expansion joints, either in the floor or slope, were put in, on account of the comparatively heavy reinforcement of the slab and the low range in temperature to which the concrete would be subjected. Fig. 10 is a plan of the reservoir, and also shows some details of the reinforcement.

The concrete for the floor lining was mixed in the following proportions, based on a mechanical analysis of the aggregates, and combining the curves, as explained by William B. Fuller and Sanford E. Thompson, Members, Am. Soc. C. E.*:

1 part cement,
3 parts sand,
1½ parts of ½-in. crushed rock,
2½ parts of 1-in. crushed rock.

This mixture worked well on the floor, but was found too rocky for the slope, as it was very difficult to tamp the concrete and prevent it

* "The Laws of Proportioning Concrete", *Transactions*, Am. Soc. C. E., Vol. LIX, p. 67.

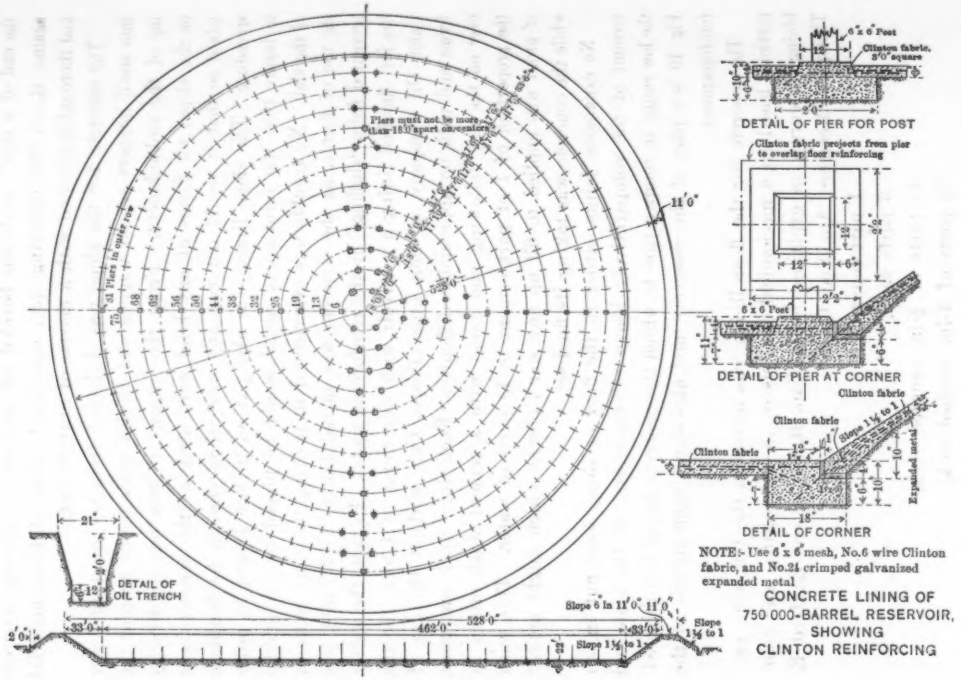

Piers must not be more than 18'0" apart on centers
11'0"
528'0"
462'0"
33'0"
2'0"
Slope 6 in 11'0"
Slope 1¼ to 1
DETAIL OF OIL TRENCH
6"x6" Post
Clinton fabric, 5'0" square
DETAIL OF PIER FOR POST
Clinton fabric projects from pier to overlap floor reinforcing
DETAIL OF PIER AT CORNER
Clinton fabric
Slope 1¼ to 1
Expanded metal
DETAIL OF CORNER
NOTE:- Use 6"x6" mesh, No.6 wire Clinton fabric, and No.24 crimped galvanized expanded metal
CONCRETE LINING OF 750 000-BARREL RESERVOIR, SHOWING CLINTON REINFORCING

FIG. 10.

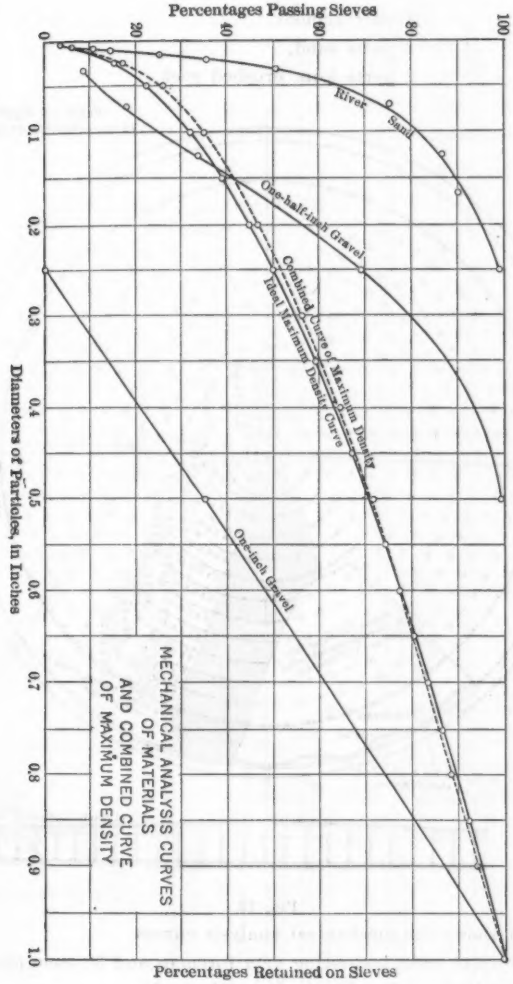

Percentages Passing Sieves
0
20
40
60
80
100
River Sand
One-half-inch Gravel
Combined Curve of Maximum Density
Ideal Maximum Density Curve
One-inch Gravel
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
Diameters of Particles, in Inches
Percentages Retained on Sieves
MECHANICAL ANALYSIS CURVES
OF MATERIALS
AND COMBINED CURVE
OF MAXIMUM DENSITY

Fig. 11.

from sluffing to the bottom of the slope. For this reason the 1-in. rock was eliminated, and the following mixture used:

1 part cement,
2 parts sand,
2 parts ½-in. crushed rock.

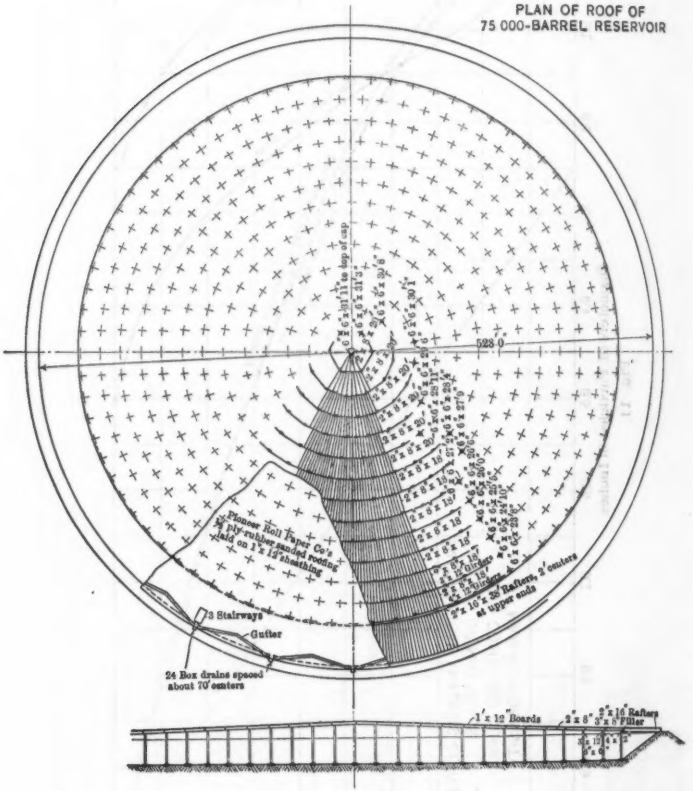
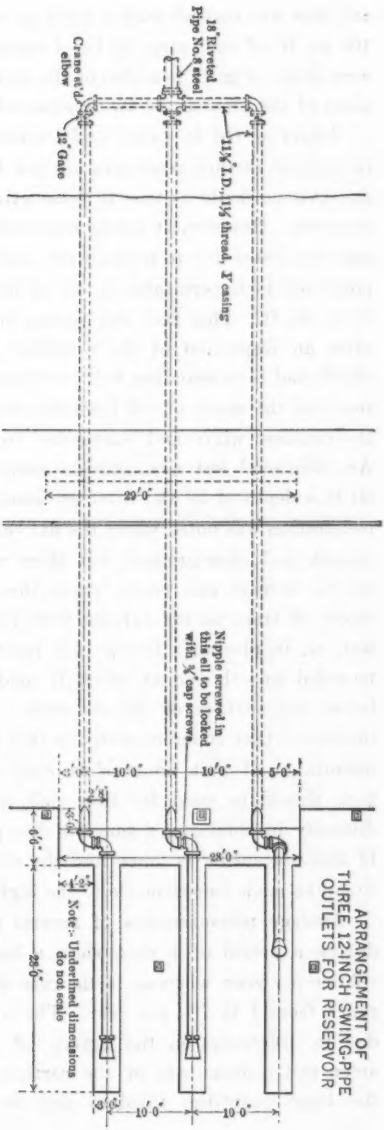


FIG. 12.

Fig. 11 shows the mechanical analysis curves.

The concrete crew, consisting of a foreman and 30 men, poured an average of 20 000 sq. ft. of lining per 9-hour day.

Roofing.—After the completion of the concrete lining, the roof was covered with a layer of Pioneer Paper Company's 1-ply roofing,



ARRANGEMENT OF THREE 12-INCH SWING-PIPE OUTLETS FOR RESERVOIR

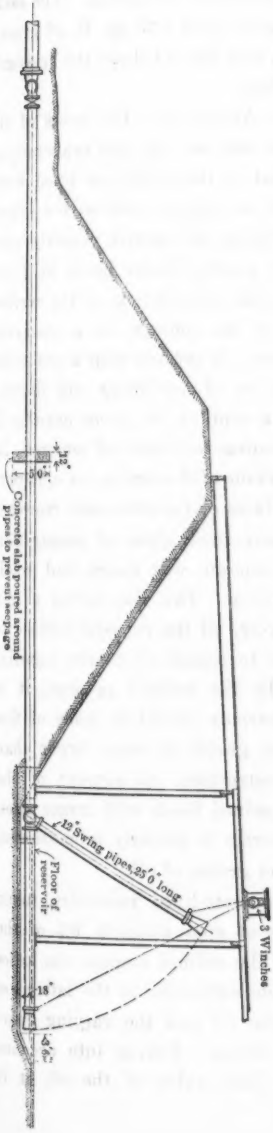


FIG. 13.

and this was covered with a layer of asphaltum and gravel. For each 100 sq. ft. of roof area, 30 lb. of asphaltum and 1.35 cu. ft. of gravel were used. Fig. 12 is a plan of the roof, and Fig. 13 shows the arrangement of the three 12-in. swing-pipe outlets.

Losses of Oil in Lined and Unlined Reservoirs.—The losses of oil in unlined earthen reservoirs are due to both seepage and evaporation, and this no doubt is true to some extent in those that are lined with concrete. However, it seems reasonable to suppose that in the latter case the losses due to seepage are negligible, as concrete properly proportioned is impermeable to oil of low gravity under heads of from 18 to 24 ft. This fact was proven to the satisfaction of the writer after an inspection of the condition of the concrete in a reservoir which had been standing full for 2 years. A scratch with a penknife removed the scum of oil from the surface of the lining and showed the concrete white and unattacked in a number of places examined. An additional test was made by suspending test slabs of concrete in oil to a depth of 20 ft.; after an immersion of 6 months, no apparent penetration was noted where the flat surfaces of the slabs were troweled smooth and close-grained, but there were some signs of penetration on the corners and edges where the concrete was rough and small pieces of stone at the surface were exposed. This was hardly a fair test, as, in placing a lining in a reservoir, all the exposed surface is troweled smooth and an effort is made to embed all coarse material below the surface of the concrete. In the writer's opinion, it is important that concrete used for this purpose should be made of fine material, and that no crushed rock or gravel in sizes larger than ¾-in. should be used for thin slab construction, on account of the difficulty in obtaining a smooth, close-grained finish with larger sizes. If this precaution is taken and the concrete is properly proportioned, it can be made impermeable to the higher grades of oil.

Average measurements of several concrete-lined reservoirs, taken during a period of 1 year, show a loss of approximately 1% of the volume per year, whereas, in the case of the unlined storage, the losses range from 1 to 7% per year. The wide variations in the latter are due to differences in the gravity of the oil and the varying character and compactness of the earthen lining. Taking into account the large quantities involved and the high value of the oil, it is

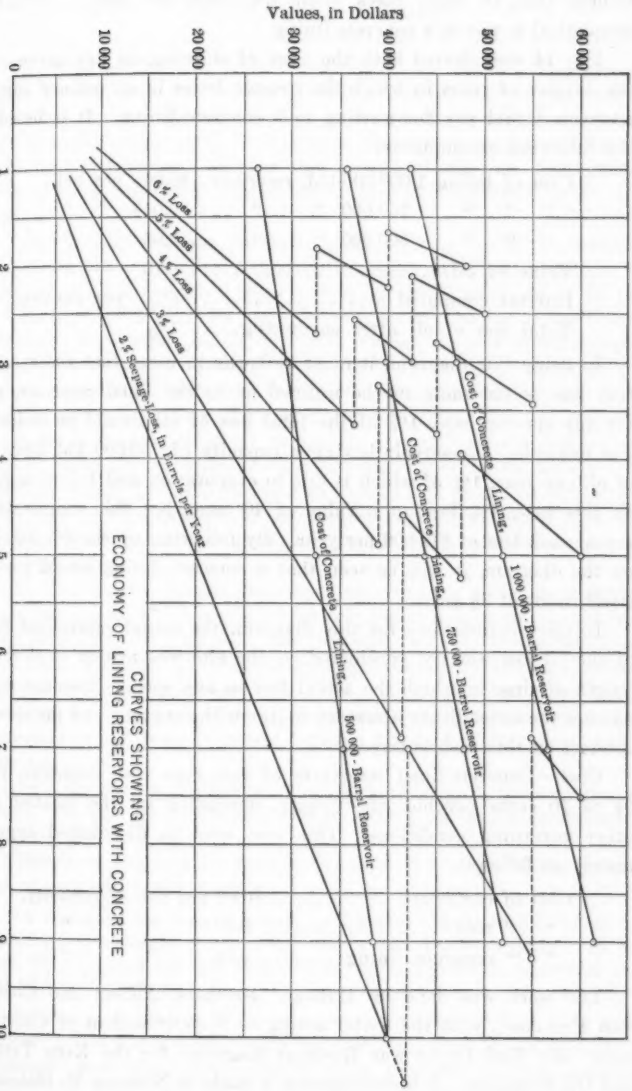


FIG. 14.—CURVES SHOWING ECONOMY OF LINING RESERVOIRS WITH CONCRETE

evident that, in many cases where the losses are large, it would be economical to put in a concrete lining.

Fig. 14 was plotted with the view of showing, in any given case, the number of years in which the seepage losses in an unlined earthen reservoir would pay for putting in a concrete lining. It is based on the following assumptions:

Cost of lining	1 000 000-bbl. reservoir	$0.04	per bbl.
" " "	750 000 " "	0.04½	" "
" " "	500 000 " "	0.05	" "
Value of oil		0.40	" "
Interest computed at		6%	per annum.
Total loss of oil after concreting		1%	" "

In using this diagram, it must be borne in mind that the evaporation loss is the same in the unlined as in the lined reservoir, and, for any specific case, 1% of the total loss of oil should be deducted. For example: A reservoir having a capacity of 750 000 bbl. loses 5% of oil per year, 1% of which is due to evaporation and 4% to seepage. In this case, 4% loss, at a value of 40 cents per bbl., represents an actual cash loss of $12 000 per year. By following up the 4% loss line on the diagram, it will be seen that a concrete lining would pay for itself in about 3½ years.

In the computations for this diagram, the actual capitalized value of the oil loss was not considered, as the aim was merely to show the length of time in which the actual loss in any specific instance would balance the expenditure necessary to line with concrete, and the accrued interest on this cash outlay.

Cost.—Concrete-lined reservoirs of this type cost, complete, from 9½ to 10 cents per bbl. of capacity, depending on the location and other governing conditions. This cost may be distributed approximately as follows:

Cost of earthwork	$0.03	per bbl. of capacity.
" " roof	0.03	" " " "
" " concrete lining	0.04	" " " "

The work was done by Mahoney Brothers, Allison and Cole, of San Francisco, with the writer acting as Superintendent of Construction. Mr. Earl Derby was Resident Engineer for the Kern Trading and Oil Company. Acknowledgment is made to Norman M. Halcombe, Assoc. M. Am. Soc. C. E., for help in preparing this paper.

DISCUSSION

C. P. Bowie,* Assoc. M. Am. Soc. C. E. (by letter).—The writer is much interested in this paper. He notes that the roof of the reservoir described was covered with roofing paper, on which was placed a layer of asphaltum and gravel. It has been his experience that such coverings, for receptacles used for the storage of oil, are very unsatisfactory and, at best, cannot be properly considered more than temporary. Mr. Bowie.

The average roofing paper is made by immersing an ordinary deadening felt (or similar material) in a solution of hot asphaltum and then passing the saturated sheet through a set of heavy rolls. In such a treatment the asphaltum not only adds to the quality of the body of the paper by toughening it, but also renders it, to a certain degree, impervious to water. The more volatile hydro-carbon compounds contained in crude oil pass off at comparatively low temperatures in the form of vapor, and their effect on these roofing papers is to "cut", as it were, the saturating material, so that in a short time the fibers of the paper, especially of that portion covering the cracks between the sheeting boards, are held together so loosely that they can be ruptured with ease by the pressure of the finger. It follows, then, that such roofings soon offer almost no resistance to the passage of gas formed by evaporation.

In the writer's opinion the proper covering for a storage reservoir, especially one in which crude oil for refining is to be kept, should be of steel plates, and "gas-tight". For this purpose, $\frac{1}{8}$-in. plates, which can be had in sizes 48 in. wide and in lengths up to 120 in., fastened together with $\frac{5}{16}$-in. rivets with $1\frac{1}{2}$-in. pitch, are ideal. In lieu of caulking, a thread weave (about $1\frac{1}{4}$ in. wide) which has previously been immersed in red lead, is placed between the laps of the sheets before riveting.

Near the center or highest point of such a roof, outlet pipes are placed, through which the gas can be drawn off, to be condensed or otherwise disposed of.

Regarding the loss by evaporation and the value of the product lost, to quote from the paper:

"A reservoir having a capacity of 750 000 bbl. loses 5% of oil per year, 1% of which is due to evaporation and 4% to seepage. In this case, 4% loss, at a value of 40 cents per bbl., represents an actual cash loss of $12 000 per year."

Assuming the losses by evaporation as stated by Mr. Cole to be correct; 1% of 750 000 bbl. would be 7 500 bbl., which product would be a material somewhat lighter than gasoline, probably between a gasoline

* San Francisco, Cal.

Mr. Bowie.

stock and casing head gas, and worth at the least consideration about 10 cents per gal., or $4.20 per bbl. The annual loss due to evaporation then would be $31 500 as compared with $12 000 lost by seepage; from which it would appear that an adequate covering for a reservoir is of more value than a concrete lining. This, of course, applies primarily to crudes which are to be used for refining purposes; however, considering the ever-increasing demand for gasoline and other refined products, it may be a matter of only a short time before necessity will require that only oils which have at least been topped, shall be sold for fuel purposes.

It is estimated that a steel roof for a reservoir costs about 2⅔ times as much as the present type of wooden roof covered with paper. Returning again to Mr. Cole's figures, this would make the cost per barrel for roof, say, 8 cents, as compared with 3 cents; or an increase of 5 cents per bbl. of capacity. The saving effected by the recovery of the oil which otherwise would be lost by evaporation, would then pay for such a roof in less than 1½ years.

Mr. Low.

EMILE LOW,* M. AM. SOC. C. E. (by letter).—The writer is very much interested in the method used in building up a compact and unyielding embankment in the construction of the oil-storage reservoirs described by Mr. Cole, and desires to record the method of building the earth embankments of the Hiland Avenue Reservoir of the new waterworks, at Pittsburgh, Pa., of which the late William H. Kennedy, M. Am. Soc. C. E., was Chief Engineer, the writer during the entire construction of this reservoir occupying various positions of responsibility.

The reservoir consists of two compartments (separated by an earth embankment), the larger one on the west and the smaller on the east side of Hiland Avenue. This street was closed and diverted, and its former site is now occupied by the division embankment. The site of the larger compartment had been occupied by market gardens, portions containing a deep, fertilized soil. The site of the smaller compartment had been covered by an apple orchard.

The reservoir is built on a comparatively level plateau overlooking the Allegheny River, just below the mouth of Negley's Run. The embankment extends around the entire periphery of the reservoir, except at the extreme western end, where, for a short distance, it is entirely in cutting.

The reservoir was to be partly in excavation and partly in embankment, and the first work was to clear the site of the embankment. In doing this, the rich top soil was first plowed, the loosened material being removed by two-horse scrapers and deposited immediately outside of the slope stakes. The underlying formation was generally clay.

* Buffalo, N. Y.

Mr. Low.

Cut-off ditches were then constructed, normal to the embankment slopes. These ditches were generally 8 ft. wide and from 2 to 4 ft. deep. They were excavated with plows and scrapers, and the removed material was placed on the cleared embankment site, which was first plowed, moistened, and rolled.

Owing to the large quantity of rich soil, much of which came from the market-garden plots, the outer slope of the embankment was built in an unusual manner. After the exact toe of the embankment had been fixed by slope stakes driven flush with the ground surface, the soil was deposited in a ridge about 2 ft. high, with a base about 6 ft. wide, carts being used for hauling.

Material from the cut-off ditches, and from the excavation inside the reservoir was placed next to this soil ridge in 6-in. layers, well watered, and rolled with a heavy cast-iron, grooved roller, drawn by two horses (or more after a rain). When the rolled embankment reached within 6 in. of the top of the ridge, more soil was deposited in a similar manner, great care being taken to have the outer surface conform to the prescribed slope, which was tested by level and measurement at frequent intervals. This method of placing the soil produced an interlocking effect. The specifications called for only 6 in. of soil, but the thickness actually averaged 12 in. or more, owing to the superabundance of this material.

The cut-off ditches previously referred to were refilled with suitable material, which was placed in thin layers, watered, and rolled. Care was taken to excavate these ditches below any roots.

In building the embankment, the lowest portions were filled first, care being taken to carry the layers up level, both longitudinally and transversely. On sloping ground the embankment was terraced or stepped, the cutting being usually 1 ft., the width depending on the slope. In transporting material from the excavation to the embankments, various methods were used. For short hauls near the embankments, two-horse scrapers were used. As the haul became longer, the scrapers were dumped into carts or wagons by drawing them over elevated timber platforms. Each platform had steep inclines and in the center there was a sliding trap door operated by hand. The filled scrapers were drawn up one incline, and after the team had passed the trap door, this was opened, the lip of the scraper was caught on a projection, and aided by the driver, this caused the scraper to turn over and deposit the material in the cart or wagon below, after which the team descended the other incline. Generally, the teams worked in a circle around each platform, and the latter was transported to another position whenever the limit of haul became too great.

Another method of excavation was effected with a machine, consisting of a large box, supported by two pairs of wheels, and drawn by three horses. This box held about 1 cu. yd. At the front end, there

Mr. Low. was a slat elevator, operated by cogs and pinions. At the bottom of the elevator there was a scoop-shaped plow. The material was loosened by this plow and then forced upon the elevator and carried into the box. When the latter was full, the scoop or plow was raised and the whole machine was hauled to the embankment.

The bottom of this box consisted of a number of hinged slats, which, when turned, allowed the material to drop out gradually, forming a layer of about 4 in. This machine was an ideal one for building the embankment, first on account of the thin and even layers deposited, and second, on account of the consolidation of the layers by the passage of the heavy loads over it. Notwithstanding this, rolling with the prescribed heavy grooved roller was never omitted, although it seemed at times to be a senseless requirement.

In excavating for the inner slope of the embankment, from 1 to 2 ft. of material was left undisturbed. As soon as the embankment reached this inner edge, the material was placed about 2 ft. inside the prescribed slope and also carried up steeper. The object of this was twofold: first, to obtain a solid embankment at the slope line; and second, to provide material for the puddle, which lined the inner slope from top to bottom.

The material on the site was generally clay, but there were quite large pockets of sand in the large compartment, and in its northern end there was also soft, sandstone rock, which latter was not allowed in the embankment, but was placed in spoil in an adjoining ravine.

Practically all the outer embankment was built of clay, a small quantity of sand being placed on the outer edge, and some intermingled with and deposited in thin layers at the same time as the clay.

In the division embankment the use of sand was allowed to a greater extent than in the outer embankment.

There was trouble from water in only one case, and this was where a spring issued near the outer edge of the northeastern embankment of the large compartment. This spring was merely walled in, and the water was carried off by an ordinary French drain.

In excavating at the south end of the eastern compartment quite a number of tile drains were unearthed. These passed under the site of the south embankment and emptied into a near-by ravine. In every case these drains were followed up and removed. The resulting excavations were roughened and widened, and then short cross-ditches were dug, all being refilled with suitable material, well moistened, and rammed by hand.

Water for moistening the embankment during construction and for other uses, was pumped from the Allegheny River, through a 1½-in. iron pipe, the pump being in one of the buildings of the Brilliant Oil Works, immediately above which the pumping station was under construction at the same time as the reservoir.

Mr. Reed.

RALPH J. REED,* ASSOC. M. AM. SOC. C. E. (by letter).—The remarkable growth of the petroleum oil industry in California during the past 15 years, shown in a most striking manner by the increase in annual production from 4 324 484 bbl. in 1900 to an estimated production of 103 000 000 bbl. in 1914, has resulted in the rapid development of facilities for handling and storing crude oils. By far the largest proportion of the crude oil stock of the State is stored in steel tanks, but there are in service, or ready for service, at the present time, 29 concrete-lined oil-storage reservoirs, having a total capacity of nearly 19 000 000 bbl. Reservoirs of this type are used largely for the storage of fuel oils varying in gravity from 14° to 18° Baumé. The lighter refining crudes, so far as the writer knows, are not stored in reservoirs, on account of the likelihood of large loss from the evaporation of the lighter and more valuable constituents, and on account of some uncertainty as to their effect on the concrete lining.

Some three or four years ago, when preparing to construct an oil reservoir, the writer had a search made in the Library of the Society for articles bearing on the subject. He was at that time unsuccessful in obtaining any information of value, and therefore welcomes Mr. Cole's paper, as it embodies a description of the latest practice in this type of construction.

It is the writer's opinion that the successful building of these reservoirs depends on a number of factors. Chief among these, as far as the earthwork is concerned, are the proper compacting of the material of the fill, and the careful building of the refill, both the bottom and the side slopes, so as to provide a uniform backing for the thin slab concrete lining. The "sheep's foot" or petrolithic rolling tamper is more successful than any other type of roller in building up a solid embankment, and the use of an ordinary road grader, to spread the material on the fill, and of a heavy disk harrow, to cut up clods of earth has proved helpful. The material of which the embankments of the reservoir at San Luis Obispo (referred to by Mr. Cole) are built, is a mixture of sand and heavy clay. The embankments built by the methods described appear to be nearly as well compacted as the ground on which they rest. The spreading of material in thin layers, and the use of as much water as possible without making the material too wet to work on, together with thorough tamping, have accomplished this result.

The method of trimming the excess material from the inner slopes, as developed by Messrs. Cole and Zeller, is extremely rapid and effective in bringing the inner slopes to line and grade with a minimum of hand work, to which it is so far superior, both in economy of labor and in results obtained, that it should be worthy of adoption on any similar earthwork where it is desired to finish a slope to accurate

* Los Angeles, Cal.

Mr. Reed.

dimensions. In order to minimize the effects of erosion on the outer slopes of the embankment, the practice of sprinkling the outer edge of the fill for a width of 6 or 8 ft. with heavy crude oil has been followed. This is done with a tank wagon equipped with a sprinkler pipe, and is carried on during the building of the fill. The oil tends to bind together the particles of material on the outer edge of the fill, and rainfall has little serious effect. Squirrels and gophers apparently dislike to burrow in oil-saturated soil, and thus the attacks of these rodents on the embankment are prevented.

It is believed to be easier and safer to erect the roof posts and girders from the floor with a derrick resting on the floor, than with a hand derrick from the fill or from the completed portion of the roof, as described by Mr. Cole. The small saw, driven either by an electric motor or a small gasoline engine, is a labor saver, when it is considered that about 220 000 ft., b. m. of sheathing, in 8 to 16-ft. lengths, must be laid, and that the number of sawn joints is large. The writer believes that Mr. Cole's statement regarding the erection of the roof by a force of 50 men in 7 days refers only to the erection of the posts, girders and rafters, which, in a reservoir of this size, would amount to about 300 000 ft., b. m. The force named would hardly be able to complete the entire roof in the time specified.

The erection of the roof prior to the pouring of the concrete lining is believed to be good practice and of importance, as it allows better control of the setting of the lining. Temperature changes are relatively small, and the lining, when carefully placed, shows little tendency even to small checks. Inspection of a recently completed reservoir, 18 days after pouring, showed practically no cracks in the bottom and about a dozen hair cracks on the entire slope. These were approximately at right angles to the intersection of the slope with the reservoir bottom, and extended up the slope from 5 to 15 ft. The writer has followed, with success, the Fuller-Thompson method of proportioning concrete, by mechanical analysis of the aggregates and construction of the curves to approximate the ideal curve, but he would use nothing larger than $\frac{3}{4}$-in. material in the mix for the floor, with $\frac{1}{4}$ to $\frac{3}{8}$-in. material for the side slopes.

Pouring the concrete lining on a 1:1 slope without forms, as Mr. Cole states, is not difficult, provided the earthen embankment has been well built, and shows no tendency to break down. The writer, however, would recommend the 1:1 inside slope only where the material to be handled contains a large percentage of clay. For embankments of sandy materials he would hesitate to recommend the 1:1 slope, but would prefer $1\frac{1}{2}$:1.

Expansion joints, of the type shown by Fig. 15, were used in the earlier reservoirs. They were constructed with a sheet of No. 20

gauge galvanized iron bent to the form shown, continuing from top to bottom of the reservoir, and placed so as to connect successive slabs of the concrete lining of the slope. In later reservoirs they have been omitted entirely, as they were in the case of the reservoir just referred to, which showed only twelve small cracks. It is the writer's belief that expansion joints are not necessary in work of this type, provided there is sufficient reinforcement in the slab and the concrete is carefully cured. Temperature variations are unusually small, especially when the reservoir is in service. Temperature observations in an empty reservoir for a short period are given in Table 1. The temperature of the oil in storage varies, during a period of one year, throughout a range of about 20°, the average temperature of oil in storage at San Luis Obispo being 60° Fahr. Mr. Reed.

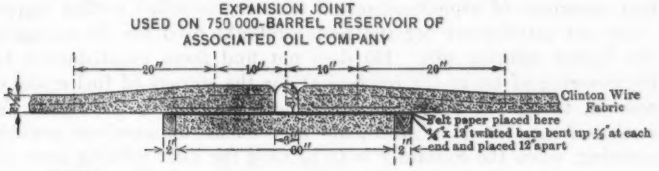


FIG. 15.

TABLE 1.—TEMPERATURES, IN DEGREES FAHRENHEIT, WITHIN AND WITHOUT RESERVOIR NO. 6 AT SAN LUIS OBISPO, CAL. (RESERVOIR EMPTY.)

December, 1915.	AT CENTER.			AT SLOPE.			OUTSIDE.		
	8.00 A. M.	12.00 M.	4.00 P. M.	8.00 A. M.	12.00 M.	4.00 P. M.	8.00 A. M.	12.00 M.	4.00 P. M.
2	62	62	66	62	62	66	60	70	64
3	60	62	62	60	62	62	58	62	68
4	58	60	64	58	60	64	50	54	56
6	54	62	66	54	62	66	46	68	62
7	56	62	67	56	62	67	44	71	60
8	55	63	64	55	63	64	42	55	53
9	59	61	64	59	61	64	49	60	54
10	52	63	62	52	63	62	39	62	52
11	51	58	63	51	58	63	44	60	55
13	58	61	64	58	61	64	52	63	56
Average.....	56.5	61.4	64.2	56.5	61.4	64.2	48.4	62.5	58.0
Daily average...	60.7			60.7			56.3		

Mr. Reed. The joint between the bottom and slope of reservoirs of this type has usually been constructed as shown in the illustrations accompanying Mr. Cole's paper. This is not designed as an expansion joint, as the reinforcing is carried through from slope to bottom, but it forms a convenient line to which to pour. As the writer believes that the small wooden form, of L cross-section, does not need to be left in place, he removes it before pouring the slope concrete.

With reference to the swing pipes on the inner ends of the outlets, it may be said that these are commonly used both in oil-storage tanks and in reservoirs. They allow the removal of oil from any level, and avoid the withdrawal with the oil of any water which may have collected at the bottom of the reservoir. They are incidentally of use when repairs are to be made to the controlling gate-valves.

The writer is inclined to agree with the statement of Mr. Bowie, that coverings of asphalt-saturated felt—the so-called roofing "paper"—are not satisfactory for the roofs of tanks used for the storage of the lighter refining oils. He does not find them unsatisfactory for the covering of tanks (or reservoirs) for the storage of fuel crudes or residua, though he admits that they are not usually gas-tight. For steel tankage, the $\frac{1}{8}$-in. steel-plate roof makes an excellent gas-tight covering, when the container is to be used for light refining crude oil, though, in his opinion, caulking the bevel-sheared edges of the plates is preferable to the insertion of a thread gasket saturated with red lead between the laps of the sheets. As was stated earlier in the discussion, few, if any, concrete-lined reservoirs are used for the storage of oils having volatile constitutents which are as valuable as those assumed by Mr. Bowie in his computation of the loss from evaporation.

Mr. Cole. E. D. COLE,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer must admit that Mr. Bowie is right in his contention that a wooden roof covered with ordinary roofing paper is rather poor construction when one considers the pains taken with the earthwork and concrete lining to make the reservoir oil-tight. However, as reservoirs of this type are intended at present only for storing fuel oil, and not the lighter grades which are used for refining purposes, the actual value of the oil lost by evaporation would be simply its market value as fuel. The writer believes that the lighter grades of oil can be stored successfully in concrete reservoirs, and that, eventually, this will be done, on account of the more economic type of construction. Then the roof specifications will no doubt be changed to include a covering which will be practically gas-tight.

* Santa Ana, Cal.

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Paper No. 1361

THE ECONOMICAL TOP WIDTH OF NON-OVERFLOW DAMS*

By William P. Creager, M. Am. Soc. C. E.

With Discussion by Messrs. Orrin L. Brodie, Edward Wegmann, and H. L. Coburn.

Synopsis.

This paper treats of the most economical width of top of the commonly-accepted type of section of solid, gravity, non-overflow dams. The writer believes it to be the general opinion of engineers that the section with a zero top width, namely, a triangular section, contains the minimum area consistent with fixed assumptions; and that the adoption of a definite width of top for a roadway or other purpose is made at a sacrifice of economy. Presumably, for this reason, many dams have been built with tops as narrow as 5% of the height. This investigation, however, shows, that the most economical width of top, for usual designing assumptions, is not zero, but lies generally between 10 and 17% of the height, according to the assumptions used in the design. As the difference in the volumes of sections, having quite a wide range of top widths, is very small compared with the uncertainty of many of the designing assumptions, the writer feels that many of his readers may consider this paper of academic rather than of economic interest.

* Presented at the meeting of January 5th, 1916.

It is just this point in particular, however, which the writer wishes to bring out, namely, that these investigations, as far as they have been carried, indicate:

1.—That there is little or no economy in the adoption of extremely narrow tops; and

2.—That exceptionally wide tops may be used, if desired, with comparatively little sacrifice of economy.

ASSUMPTIONS.

The curves shown on Fig. 1 cover seven sections, designed under different assumptions. They indicate for each the most economical width of top, in terms of height, and the relative areas of sections having other top widths. The assumptions used have been designated by letters, and are as follows:

Location of Resultant.—

A.—Resultant, reservoir full, to intersect all horizontal joints at the exact extremity of the middle third; except near the top before the down-stream face departs from the perpendicular, where the resultant lies within the middle third.

B.—Same as *A*, except to intersect at a point within the middle third a distance equal to one-fifteenth of the width of the joint.

C.—Resultant, reservoir empty, to intersect all horizontal joints at the exact extremity of the middle third; except near the top, before the up-stream face departs from the perpendicular, where the resultant lies within the middle third.

D.—Resultant, reservoir empty, to have no influence on the design of the section.

Forces Considered.—

E.—Weight of concrete; assumed specific gravity, 2.25.

F.—Weight of concrete; assumed specific gravity, 2.33.

G.—Horizontal component of water pressures.

H.—Vertical component of water pressures on the battered up-stream face.

I.—Horizontal silt pressure; silt assumed to be a liquid with a specific gravity of 0.64 in addition to the water pressure. Depth of silt five-tenths of the height of the section.

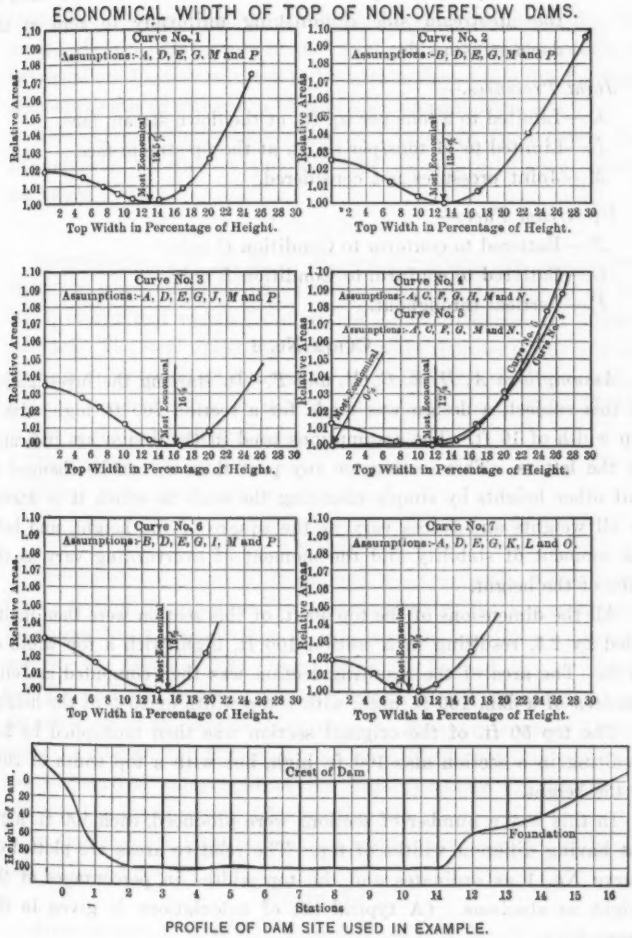
ECONOMICAL WIDTH OF TOP OF NON-OVERFLOW DAMS.
Curve No. 1
Assumptions:-A, D, E, G, M and P
Most Economical 13.5%
Relative Areas.
Top Width in Percentage of Height.
Curve No. 2
Assumptions:-B, D, E, G, M and P
Most Economical 13.5%
Relative Areas.
Top Width in Percentage of Height.
Curve No. 3
Assumptions:-A, D, E, G, J, M and P
Most Economical 16%
Relative Areas.
Top Width in Percentage of Height.
Curve No. 4
Assumptions:-A, C, F, G, H, M and N.
Curve No. 5
Assumptions:-A, C, F, G, M and N.
Most Economical 0%
Most Economical 12%
Curve No. 5
Curve No. 4
Relative Areas.
Top Width in Percentage of Height.
Curve No. 6
Assumptions:-B, D, E, G, I, M and P
Most Economical 13%
Relative Areas.
Top Width in Percentage of Height.
Curve No. 7
Assumptions:-A, D, E, G, K, L and O.
Most Economical 9%
Relative Areas.
Top Width in Percentage of Height.
Crest of Dam
Foundation
Height of Dam.
Stations
PROFILE OF DAM SITE USED IN EXAMPLE.

FIG. 1.

J.—Uplift on all horizontal joints. Total uplift assumed to be represented by a triangle, the unit uplift equal to five-tenths of the hydrostatic pressure due to the total head of water at the up-stream side diminishing uniformly to zero at the down-stream side.

Joint Pressures.—

K.—Limited to 9 tons per sq. ft., at the down-stream face.

L.—Limited to 11 tons per sq. ft. at the up-stream face.

M.—Joint pressures not considered.

Up-stream Face.—

N.—Battered to conform to Condition *C* only.

O.—Battered to conform to Condition *L* only.

P.—Vertical throughout.

CURVE NO. 1.

Assumptions A, D, E, G, M, and P.—In starting the investigation of this subject, a design was made for a section 200 ft. high, with a top width of 10 ft. The assumptions used in the design are indicated by the letters. This section, or any part thereof, can be changed to suit other heights by simply changing the scale to which it is drawn, as all weights and forces vary as the square of the height, and both the moment of stability and the moment of overturning vary as the cube of the height.

All the dimensions of the top 40 ft. of this section were then multiplied by 2.5, resulting in a section 100 ft. high, with a top width of 25 ft. The area of the resulting section was then computed as being the area of a dam 100 ft. high, with a top width of 25% of the height.

The top 50 ft. of the original section was then multiplied by 2.0, resulting in a section also 100 ft. high, but with a top width of 20% of the height.

In this way a number of sections were produced, each 100 ft. high, but having different widths of top. The relative areas are plotted on Curve No. 1 as ordinates and the top widths in percentages of the height as abscissas. (A typical set of calculations is given in the Appendix.)

It will be noted from this curve that the most economical top width of sections designed in accordance with these assumptions, is about 13.5% of the height.

CURVE No. 2.

Assumptions B, D, E, G, M, and P.—Curve No. 2 was prepared in the same manner and under the same assumptions as adopted for Curve No. 1, except that the resultant, reservoir full, instead of intersecting all horizontal joints at the exact extremity of the middle third (Assumption *A*), was required to lie within the middle third a distance of one-fifteenth of the width of the joint (Assumption *B*).

This curve, though slightly different in shape from Curve No. 1, also indicates the most economical width of top to be about 13.5% of the height.

CURVE No. 3.

Assumptions A, D, E, G, J, M, and P.—Curve No. 3 was derived under the same assumptions as those governing the design of Curve No. 1, except that uplift (Assumption *J*) at all horizontal joints was included. The total uplift was assumed to be represented by a triangle, the unit uplift equal to five-tenths of the hydrostatic pressure due to the total head of water at the up-stream side diminishing uniformly to zero at the down-stream side.

This curve indicates the most economical width of top for these conditions to be about 16% of the height.

CURVE No. 4.

Assumptions A, C, F, G, H, M, and N.—Curve No. 4 was prepared from a section, also designed in accordance with the assumptions used for Curve No. 1, except that in this case the specific gravity of the concrete was taken at 2.33 (Assumption *F*), instead of 2.25 (Assumption *E*), and the resultant was also required to intersect the base at the extremity of the middle third when the reservoir is empty (Assumption *C*). Assumption *C* necessitated a slightly battered up-stream face and the vertical component of the water pressure on this face was added to the forces acting (Assumption *H*).

In this case the most economical width of top appears to be about 12% of the height.

CURVE No. 5.

Assumptions A, C, F, G, M, and N.—Curve No. 5 is the same as Curve No. 4, except that the vertical component of the water pressure on the battered up-stream face (Assumption *H*) was neglected. In this case, alone, the most economical width was found to be zero,

but the curve is seen to be nearly horizontal between 0% and 14%, at the latter width involving an increase of material of only ½ of 1 per cent. This curve will apply directly to "Theoretical Type No. 2," by Edward Wegmann, M. Am. Soc. C. E., as it will be noted that the assumptions covering the design of the section are exactly the same as those used by Mr. Wegmann. It might be remarked here that the condition of requiring the resultant to lie within the middle third with reservoir empty is often omitted by engineers, a vertical up-stream face being adopted, unless a batter is required for the condition of limiting toe pressures.

Curve No. 6.

Assumptions B, D, E, G, I, M, and P.—Curve No. 6 was based on the assumptions used in preparing Curve No. 2, except that in this case silt pressure (Assumption *I*) was included. The silt pressure was assumed to be a liquid with a specific gravity of 0.64 in addition to the water pressure, and its depth was assumed as five-tenths of the height of the section.

On account of the silt pressure, the expedient of changing the scale, resorted to in computing previous curves, would apply from the top of the dam to the surface of the silt only. The rest of each section had to be computed separately for each point on the curve.

Assumption *I* seems to lead to a slightly larger economical top width, appearing on the curve to be about 15% of the height.

Curve No. 7.

Assumptions A, D, E, G, K, L, and O.—Thus far there has not been taken into consideration the condition of limiting joint pressures (Assumptions *K* and *L*). Curve No. 7 is based on these assumptions. In other respects the assumptions used were the same as for Curve No. 1.

On account of Assumptions *K* and *L*, the expedient of changing the scale, resorted to in computing Curves Nos. 1 to 5, would apply only from the top of the dam to the elevation at which the limiting joint pressures began to govern the design. The rest of each section had to be computed separately for each point on the curve.

The vertical component of the water pressure on the battered up-stream face (Assumption *H*) was neglected in order to simplify

the calculations. A comparison of sections 200 ft. high was also used in computing Curve No. 7.

For this curve the most economical width was found to be about 9% of the height. In all probability, to include in the calculations the vertical component of the water pressure on the battered up-stream face would increase the most economical width, as it was seen from Curves Nos. 4 and 5 that it resulted in an increase from 0 to 12 per cent.

Method of Application.

It must be remembered that the curves apply only to dams of constant height throughout their length. In order to obtain the greatest economy, the top width, theoretically, should be a fixed percentage of the height at any point. As a varying width of top is objectionable, for many reasons, a constant width should be adopted which will be somewhat less than that corresponding to the most economical for the maximum height, the amount of such reduction depending on the relative quantity of material contained in that portion of the dam less in height than the maximum.

In order to indicate the amount of such reduction the writer has designed a dam for the profile indicated on Fig. 1 in accordance with the assumptions used in computing Curve No. 3, and found the most economical top width for this dam to be 14% as compared with 16% indicated by Curve No. 3 for the maximum section (100 ft.).

This indicates that very little reduction in top width is necessary unless the variation in height of dam at different points along the profile is considerable.

Conclusion.

The assumptions used herein cover in a general way most of the important conditions usually considered, with the exception of ice thrust. However, as the consideration of overturning forces in addition to the water pressure seems to increase the most economical top width, as in the case of uplift and silt conditions (Curves Nos. 3 and 6); and as the consideration of ice thrust, in itself, increases greatly the top part of the section, it seems logical to assume that an economical top width for ice thrust condition would be at least as great as that indicated in Curves Nos. 3 and 6.

It is believed, therefore, that, except for Curve No. 7 (which, however, would probably have been similar to the rest if the vertical

component of the water pressure had not been neglected), practically no economy results in selecting a top width for dams of practically uniform height less than about 14% of the height; and that, for some designing assumptions, a width of even 17% involves no sacrifice of economy.

It is true that the assumptions on which these conclusions are based do not consider sliding or vertical shear. It is believed, however, that cases where these considerations affect the shape of the section are the exception rather than the rule. Moreover, in the light of these investigations, as far as they have gone, it is hard to say whether these conditions would require a smaller or a larger top width than indicated in the curves.

The writer regrets that he has not had the time to carry these investigations farther.

APPENDIX

Calculations for Curve No. 3.

Assumptions A, D, E, G, J, M, and P.—

Let:

H = The height of the dam above any horizontal joint, the length of which is to be determined;

L = The length of a horizontal joint to be determined;

L_0 = The known length of the horizontal joint next above;

h = The vertical distance between these two horizontal joints;

w = The weight of 1 cu. ft. of masonry = 140.5 lb.;

A = The resultant weight of the dam, above the known horizontal joint, divided by w; it is also equal to the area;

m = The horizontal distance from the heel of the dam to the resultant A;

P = The total water pressure on the up-stream face of the dam divided by w;

U = The total uplift pressure on the unknown joint divided by w;

s = The specific gravity of the masonry = 2.25;

M_P = The moment, about the heel, of water pressure on the up-stream face of the dam divided by w;

M_U = The moment, about the heel, of the uplift divided by w;

M_A = The moment, about the heel, of the weight of the dam divided by w.

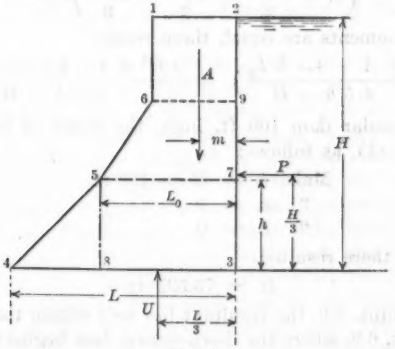

Fig. 2.

In the calculations, the weight of 1 cu. ft. of masonry was taken as unity; therefore the weight of 1 cu. ft. of water would be $\frac{1}{s} = \frac{1}{2.25}$.

The method of design, as worked out by Messrs. Morrison and Brodie, Wegmann, and others, was used.

A general formula to suit this particular case was first derived.

In Fig. 2 suppose the dam to have been already designed, down to the joint, 5-7. A general formula for the length, L, of the joint, 4-3, was derived as follows:

The weights and moments acting on the dam are given in Table 1.

TABLE 1.—WEIGHTS AND MOMENTS.

Portion.	Weight.	Moment about Point 3.
Masonry above joint 5-7.....	$+A$	$+A\,m$
Masonry in 5-7-3-8.....	$+L_0\,h$	$+\frac{L_0^2\,h}{2}$
Masonry in 4-5-8.....	$+\frac{L\,h}{2}-\frac{L_0\,h}{2}$	$+\frac{L^2\,h}{6}+\frac{L\,h\,L_0}{6}-\frac{L_0^2\,h}{3}$
Uplift, U.....	$-\frac{L\,H}{9}$	$-\frac{L^2\,H}{27}$
Horizontal water pressure, P..	Zero	$+\frac{H^3}{13.5}$

The total moment about Point 3 is:

$$A\,m+\frac{L_0^2\,h}{6}+\frac{L^2\,h}{6}+\frac{L\,h\,L_0}{6}-\frac{L^2\,H}{27}+\frac{H^3}{13.5}.$$

The reaction of the foundation is equal to the net weight of the dam, and the moment of the foundation reaction about Point 3 is the reaction multiplied by $\frac{2\,L}{3}$, or

$$\left(A+\frac{L_0\,h}{2}+\frac{L\,h}{2}-\frac{L\,H}{9}\right)\frac{2\,L}{3}.$$

As these two moments are equal, there results:

$$L^2+L\left(\frac{18\,A+4.5\,h\,L_0}{4.5\,h-H}\right)=\frac{2\,H^3+4.5\,L_0^2\,h+27\,A\,m}{4.5\,h-H}\ldots.(1)$$

For a triangular dam 100 ft. high, the width of base was found from Equation (1), as follows:

Make $h = H = 100$ ft.
" $A = 0$
" $L_0 = 0$

Solving for H, there resulted:

$$H = 75.592 \text{ ft.}$$

Above the joint, 6-9, the resultant lies well within the middle third, and at the joint, 6-9, where the down-stream face begins to depart from the vertical, it is just at the extremity of the middle third. In order to find the height, 1-6, of this portion, the following assumptions for substitution in Equation (1) were made:

$A = 0$
$L_0 = L = 10$ ft., the width of top,
$h = H$

Solving for H, there resulted:

$$H = 13.229 \text{ ft.}$$

To find the length, L, of a joint 16 ft. from the top, the dam was assumed to have been designed, as indicated above, down to an elevation 13.229 ft. from the top. For this case:

$$\begin{aligned} L_0 &= 10 \\ A &= 13.229 \times 10 = 132.29 \\ m &= \frac{10}{2} = 5 \\ H &= 16 \\ h &= 16 - 13.229 = 2.771 \end{aligned}$$

With these substitutions in Equation (1), there resulted: $L = 11.10$ ft. The area and all other characteristics of the section from the top to an elevation 16 ft. from the top were then calculated. For the length of the joint 20 ft. from the top, the following substitutions in Equation (1) were made:

$$\left.\begin{aligned} L_0 &= 11.10 \\ A &= 161.52 \\ m &= 5.280 \end{aligned}\right\} \text{(from the foregoing)}$$

$$\begin{aligned} H &= 20 \\ h &= 20 - 16 = 4 \end{aligned}$$

With these substitutions, there resulted: $L = 13.06$ ft.

In this way, the whole section was designed by successive steps to an elevation 200 ft. from the top. A summary of the calculations is given in Table 2.

The next step was to derive from Table 2 a number of other sections, each 100 ft. high, but having different top widths. Columns 1 and 6 of Table 2 are repeated on Lines 1 and 2 of Table 3. A section of this type of any height can be adapted to any other height by simply changing the scale to which it is drawn, as the weights and water pressures are all functions of H^2, and the moments of the weight of masonry and water pressures are functions of H^3. This was done in Table 3. The heights and top widths indicated on Lines 1 and 3 were multiplied by the factors on Line 4, and entered on Lines 6 and 8, respectively. The new heights were all 100 ft.

The areas on Line 2 were multiplied by the square of the multiplying factor to give the new areas entered on Line 7.

The top width, as a percentage of the height given on Line 9, was obtained by dividing the quantities on Line 8 by the corresponding quantities on Line 6 and multiplying by 100 to obtain the percentages.

Calling the minimum area on Line 7 equal to 1.0, the other areas, in terms of this one, were found and entered on Line 10.

The quantities from Line 10, as ordinates, and the corresponding quantities from Line 9, as abscissas, were used in plotting Curve No. 3.

TABLE 2.—Summary of Calculations for a Dam 200 Ft. High and a Top Width of 10 Ft.
Assumptions *A*, *D*, *E*, *G*, *J*, *M*, and *P*.

(1) Height.	(2) P	(3) M_P	(4) U	(5) M_U	(6) A	(7) M_A	(8) L
13.229	38.890	171.49	14.70	49.00	132.29	661.45	10.00
16	56.888	308.40	19.73	73.00	161.52	815.76	11.10
20	88.888	592.59	29.02	126.26	209.84	1 108.2	13.06
25	138.88	1 157.4	44.81	240.92	282.81	1 642.6	16.13
30	200.00	2 000.0	65.47	428.83	372.24	2 444.9	19.64
40	355.55	4 740.7	121.64	1 109.7	607.29	5 231.7	27.37
50	555.55	9 259.2	196.98	2 327.9	921.42	10 193	35.46
60	800.00	16 000	290.27	4 211.8	1 316.2	18 018	43.54
70	1 088.8	25 407	401.57	6 911.0	1 792.1	29 367	51.63
80	1 422.2	37 925	529.87	10 528.5	2 348.3	44 862	59.61
100	2 222.2	74 074	836.89	21 014	3 697.6	90 582	75.32
150	5 000.0	250 000	1 889.8	71 428	8 415.3	316 172	113.77
200	8 888.8	592 592	3 359.5	169 299	15 029	756 664	151.18

TABLE 3.—Determination of the Co-Ordinates Used in Plotting Curve No. 3.

1. Height from Table 2	30	40	50	60	70	80	100	150	200	100*
2. A " " 2	372.24	607.29	921.42	1 316.2	1 792.1	2 348.3	3 697.6	8 415.3	15 029	3 779.6
3. Top " " 2	10	10	10	10	10	10	10	10	10	0
4. Multiplying factor	10	10	10	10	10	10	10	10	10	10
5. " " squared	3 / 100 / 9	4 / 100 / 16	5 / 100 / 25	6 / 100 / 36	7 / 100 / 49	8 / 100 / 64	10 / 100 / 100	15 / 100 / 225	20 / 100 / 400	10 / 100 / 100
6. New height	100	100	100	100	100	100	100	100	100	100
7. New A	4 186.0	3 795.5	3 685.6	3 656.3	3 657.4	3 668.0	3 697.6	3 740.1	3 757.4	3 779.6
8. New top	33.33	25.00	20.00	16.67	14.29	12.50	10.00	6.67	5.00	0
9. New top width, in percentage of height	33.33	25.00	20.00	16.67	14.29	12.50	10.00	6.67	5.00	0
10. Relative areas	1.134	1.0382	1.0081	1.0000	1.0004	1.0033	1.0114	1.0230	1.0277	1.0338

* Triangular dam.

DISCUSSION

Mr. Brodie.

ORRIN L. BRODIE,* M. AM. SOC. C. E.—The author has indicated the range in the economic top width of non-overflow dams as varying from about 10% to a little more than 17% of the height, according to the assumptions of design. Such discussion, if academic, as the author fears, is not without value if it helps in any way to fix the ideas.

The lower limit of 10% has been recognized for some time as of practical value; but, in connection therewith, the factors affecting width, besides pure economy, should not be neglected. The speaker refers to requisite width of roadway across the top of the dam, drainage wells within the body of the structure, and upper inspection gallery in a longitudinal direction, all of which require space which in large part often governs in fixing a top width. Incidental to some of the above should be mentioned the architectural treatment of railing or parapet, which is also closely related to the coping and cove stones or blocks, space for which must be provided at the top of the dam and so as not to encroach on the interior passageways, if any, at the top.

The author expresses a belief that the prevailing opinion of engineers is that the triangular dam (presumably with the apex at the water surface) provides the minimum section for fixed assumptions. Although the triangular section is theoretically fitted to withstand a horizontal liquid pressure intensity, increasing downward from zero at the top, the speaker seriously doubts that such general opinion prevails, for, unless he is much mistaken, it must be obvious to any one having to deal with the design of dams, especially high ones, that there is no economy in a triangular profile, in all cases, over one with a wider top, for the superimposed weight of the latter would result in correspondingly shorter bases in the lower portion than would obtain for a triangular cross-section. Furthermore, where uplift is considered active, it should be remembered that its tendency to overturn the structure is a function of the length of base.

The speaker noted the omission of consideration of ice pressure in the studies presented, except for passing mention. A study of its effect on the percentages deduced, should prove interesting.

As nothing is stated in the paper concerning the superelevation of the top of the dam, it is to be doubtless inferred that the top in each case was taken flush with the water surface.

At the risk of being tedious, perhaps, the speaker will continue his discussion, also in a way that may be said to be somewhat academic; but, to illustrate what effect on the cross-section consideration of ice pressure and of superelevation of the top has, and to compare ordinary sections with the triangular, four cross-section designs, *A*, *C*, *F*, and

* New York City.

Mr. Brodie.

D, have been prepared, the base of each of which is subject to a head of 100 ft., the top in every case except the triangular dam, *D*, being 10 ft. wide.

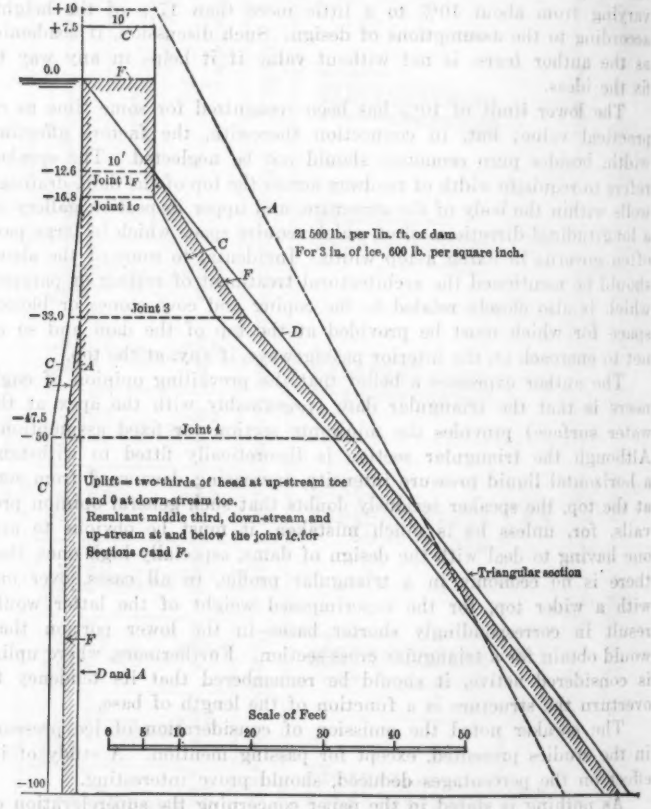

FIG. 3.

The method of design is identical with that outlined previously* by the speaker. In fact, two of the cross-sections were taken from the earlier discussion and extended.

A is designed for 21 500 lb. per lin. ft. of dam, ice pressure at water surface; uplift intensity in joint of two-thirds of the head at the

* In discussion of "Provision for Uplift and Ice Pressure in Designing Masonry Dams", by the late C. L. Harrison, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 174.

Mr. Brodie.

up-stream edge of each joint, diminishing to zero at the down-stream edge, and the horizontal thrust of the water. The relative density of the masonry is assumed at 2¼. The resultant is nowhere outside of the middle-third limit of the joint in question.

The same conditions, with the exception of ice pressure, apply to the other three sections.

Section *A*, as brought out in the discussion previously referred to, required a superelevation and top width of at least 10 ft., that is, no practically satisfactory cross-section resulted from the analytic design with lower values for the top width and superelevation.

Section *C* was assumed to have a superelevation of 7.5 ft., and Sections *F* and *D* have their tops at the reservoir surface.

In comparing these sections, note how the down-stream face of the triangular section (*D*'s) is everywhere down stream or exterior to the respective faces of Sections *F* and *C*. Similarly, *F*'s face is exterior to *C*'s. This shows, in part, the effect on the cross-section of the superelevation (*C*) and sensible top width of the dam (*F*).

With regard to the up-stream faces, however, it is to be noticed that where *C*'s down-stream face was the innermost, it is now the outermost, with respect to the sections. *F* is still intermediate with respect to the positions of *D*'s and *C*'s up-stream faces.

The ice-pressure section, it will be observed, encroaches within the other sections toward the bottom.

Table 4 gives the areas and bases, etc., of the cross-section for depths of 50 and 100 ft.

TABLE 4.

Cross-section.	Conditions.	50 Ft. Depth of Water.		100 Ft. Depth of Water.			
		Area, in square feet.	Base, in feet.	Area, in square feet.	Top width, in feet.	Superelevation, in feet.	Base, in feet.
A.....	Ice, horizontal and upward.	.....	....	4 375	10	10	69.0
C.....	Horizontal and upward.....	999.0	39.5	3 969	10	7.5	79.3
F.....	Horizontal and upward.....	998.0	40.0	3 973	10	0	79.0
D.....	Horizontal and upward.....	992.5	39.7	3 970	0	0	79.4

Ice = 21 500 lb. per lin. ft. of dam.
Uplift = two-thirds of the head at the up-stream edge of the joint, decreasing uniformly to zero at the down-stream edge.
Specific gravity of masonry = 2¼.

Turning attention now to the quantities of masonry involved, the result presents itself that, save for the ice-pressure section, *A*, all the sections are practically of the same volume, therefore it may be concluded that, for a given width, the superelevation affects, not the quantity of material, but only its distribution in the cross-section.

The quantity of material in the cross-section would seem to depend on the top width, to a certain extent; but more on the forces assumed

Mr. Brodie. to be acting on the dam. Although the superelevation thus has its effect on the shape of the cross-section, the practical questions of rise of reservoir due to flood, wave height, etc., determine in large part what superelevation is desirable.

Mr. Wegmann. EDWARD WEGMANN,* M. AM. SOC. C. E. (by letter).—The author has discovered the interesting fact that, for a dam of the type shown in Fig. 4, there is a certain top width which gives the most economical section. One would suppose that the narrower the top width, the smaller would be the area of the profile. Mr. Creager has proved that such is not the case, and the method by which he found the top width giving the most economical profile, is correct, provided he does not insist on a rigid compliance with the usual requirement that the lines of pressures, reservoir full or empty, shall be kept within the center-third of the profile.

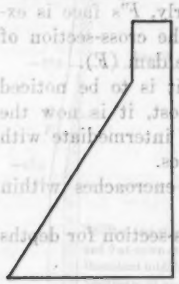

FIG. 4.

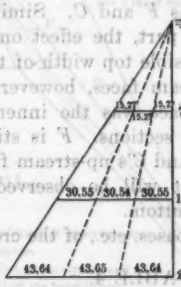

FIG. 5.

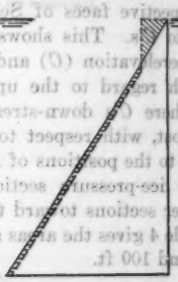

FIG. 6.

Fig. 5 shows a triangular profile without any top width, which is designed to resist solely the hydrostatic pressure of the water, no attention being paid to shocks from floating bodies or from waves. Upward pressure is not considered in this case. In this profile the lines of pressure, for reservoir full and empty, are, respectively, on the down-stream and up-stream limits of the center-third of the profile, and the latter has, therefore, the minimum area which will comply with the given conditions.

If, however, a thin slice of masonry is cut from the down-stream face of the dam, and placed as a small inverted triangle at the top of the dam, as shown by the shaded area in Fig. 6, it will be found that the dam has been strengthened, even though the width of its base has become slightly less, as the moment around its toe is greater for the small inverted triangle than for the thin slice of masonry. In this case, therefore, there is a surplus of strength, and the area of the profile can be reduced slightly.

* New York City.

Mr. Wegmann.

There is, however, a limit to the thickness of the slice of masonry which can be transferred from the front face to the top of the dam, as Mr. Creager has shown. The top width which gives the most economical profile can probably be determined by the differential calculus.

Although it is true that one can obtain, in this manner, a profile (Fig. 6), which has less area than the simple triangular profile (Fig. 5), it does not comply strictly with the given condition for reservoir empty. The effect of the small inverted triangle placed at the top of the dam, is to draw the line of pressure, reservoir empty, slightly outside of the center-third. The extent to which this is done is of no practical importance, but the fact remains that, for the given conditions, the triangular profile (Fig. 5) has the minimum area.

If Mr. Creager's rule for the least top width is applied in the design of a dam, this width will have to vary from one end of the dam to the other, according to the height of the structure. This, of course, would never be done in actual practice. Usually, the top width of a masonry dam is not to be determined by theoretical principles, but by practical considerations, such as the strength required to resist shocks from waves or floating bodies, the breadth needed for a roadway over the top of the dam, etc.

Mr. Coburn.

H. L. COBURN,* M. AM. SOC. C. E. (by letter).—This paper was read with much interest by the writer as he had made a study of the same problem, some time previously, but along somewhat different lines. Like Mr. Creager, the writer is not sure that the presentation is of much more than academic interest, but it may give some indication as to why different designers, starting from similar premises, reach such apparently different results in designing the cross-section of a masonry dam.

The writer's method of attack is shown in the accompanying computations, and the results are given in both tabular and graphical form.

The assumptions were as follows:

Specific gravity of masonry 2.3; upward pressure over entire base, varying uniformly from that due to full hydrostatic head at heel to zero at toe; resultant of all forces, reservoir full, must cut base at one-third its length from the toe, that is, there must be no stress at extreme up-stream edge of base. This, of course, assures the minimum section allowable, if the masonry is assumed to have no tensile strength.

The section assumed is made of two right-angle triangles, the fillet which usually would be provided at the junctions of the two triangles in the down-stream face being considered as negligible.

For convenience in making the computations, the width of top was taken in terms of width of base rather than of height, and was

* New York City.

Mr. Coburn.

expressed by $b = B \div k$; the width of the base was then calculated for various values of k. It should be noted that, for all values of k greater than about 2.6, or top less than 0.4 of the base, or 0.3 of the height, there is only a small variation in the area of the cross-section, with the area a minimum, when the top is about 0.25 of the base or 0.2 of the height. For this section, the base is 0.81 of the height. The average pressure on the base, with reservoir full, is 45 H, in pounds per square foot, and the maximum at the toe is twice this, or 89 H, in pounds per square foot. With the reservoir empty, the average pressure will be 76 H; the maximum at the heel will be 157 H, and the minimum at the toe -4.5 H. Indeed, it should be noted that, with a vertical up-stream face and any width of top greater than zero and less than one-half the base, there will always be some tension at the toe when the reservoir is empty, but this will usually be negligible.

It is of interest, also, to note that, for all widths of top less than one-half, the ratio of horizontal to vertical forces is very high, indicating the necessity of great care in providing means to prevent sliding.

TABLE 5.—FUNCTIONS OF SECTION, IN TERMS OF HEIGHT, FOR VARIOUS WIDTHS OF TOP, ASSUMING UPWARD PRESSURE ON BASE VARYING UNIFORMLY FROM FULL HYDROSTATIC HEAD AT HEEL TO ZERO AT TOE.

k	r	R	C	A	P_1	P_2	P_3	P_4	F
1.0	1.0	0.877	0.877	0.877	112.50	143.75	143.75	143.75	0.317
1.2	0.83	0.615	0.738	0.623	90.58	121.79	177.03	66.54	0.458
1.4	0.71	0.508	0.712	0.539	77.31	108.52	185.64	31.58	0.567
1.5	0.67	0.474	0.711	0.516	72.56	103.79	186.48	19.16	0.604
1.6	0.63	0.445	0.712	0.495	66.68	99.96	185.83	14.03	0.640
1.8	0.56	0.400	0.720	0.472	62.81	93.78	182.72	4.93	0.690
2.0	0.5	0.365	0.730	0.459	58.63	89.86	179.69	0.00	0.730
3.0	0.33	0.262	0.787	0.436	48.63	79.85	165.04	— 5.32	0.816
4.0	0.25	0.208	0.812	0.432	45.13	76.24	157.24	— 4.48	0.854
5.0	0.20	0.166	0.831	0.432	43.50	74.75	152.96	— 3.45	0.865
6.0	0.17	0.141	0.843	0.433	42.63	73.87	150.41	— 2.66	0.870
8.0	0.13	0.107	0.856	0.435	41.75	73.00	147.67	— 1.68	0.874
10.0	0.10	0.086	0.864	0.436	41.38	72.59	146.34	— 1.15	0.875
∞	0.00	0.000	0.877	0.439	40.63	71.88	143.75	0.00	0.877

k = Ratio of base to top;
r = " " top to base = $\frac{1}{k}$;
R = " " " " height;
C = " " base to height, $B = CH$;
A = Area of cross-section, in terms of square of height, area = AH^2
P_1 = Average pressure on base, reservoir full, in terms of height, $P_{av.} = P_1 H$
P_2 = " " " " " empty " " "
P_3 = Maximum " at heel " " " " "
P_4 = Minimum " at toe " " " " "
F = Ratio of horizontal to vertical forces.

All in pounds per square foot.

Inspection of Table 5 will make it clearly apparent that, with a simple triangular section, that is, zero width of top, a dam so designed will be equally stable at all portions, regardless of height, and the stresses in the masonry and on the foundations will be a maximum

Mr. Coburn.

at the greatest section. It is equally apparent that, with a top of any considerable width, a dam, having a maximum section designed in accordance with these figures, will be least stable at its point of greatest height where the resultant will just cut the middle-third.

Consider the section as made up of two triangles, as shown in Fig. 7, the fillet at a being negligible.

Assume an upward pressure on the base equal to the full hydrostatic head at the heel, decreasing uniformly to zero at the toe.

The resultant, with reservoir full, is to cut the base at the down-stream one-third point; that is, no stress at the heel.

The problem is to find the width of the base, B, in terms of the height, H, with varying widths of the top, $b = B \div k$.

Take moments about the down-stream one-third point, as ΣM about this point must equal 0 to meet conditions imposed.

FIG. 7.

$$\text{Moment of horizontal pressure} = \frac{w\,H^2}{2} \times \frac{H}{3} = \frac{w\,H^3}{6}$$

$$\text{Moment of upward pressure} = \frac{w\,H\,B}{2} \times \frac{B}{3} = \frac{w\,H\,B^2}{6}$$

$$\text{Moment of weight of lower triangle} = \frac{2.3\,w\,H\,B}{2} \times \frac{B}{3} = \frac{2.3\,w\,H\,B^2}{6}$$

$$\text{Moment of weight of upper triangle} = \frac{2.3\,w\,h\,b}{2} \times \left(\frac{2\,B}{3} - \frac{2\,b}{3}\right) = \frac{4.6\,w\,H\,B^2}{6}\left(\frac{k-1}{k^3}\right)$$

$$\Sigma M = H^3 + H\,B^2 - 2.3\,H\,B^2 - 4.6\,H\,B^2\left(\frac{k-1}{k^3}\right) = 0$$

$$B = H\left(1.3 + 4.6\,\frac{k-1}{k^3}\right)^{-\frac{1}{2}} = C\,H \ldots\ldots\ldots\ldots (1)$$

Second.—Area of cross-section in terms of height:

$$A = \frac{1}{2}(B\,H + b\,h) = \frac{1}{2}\left(C\,H^2 + \frac{C\,H^2}{k^2}\right) = \frac{C\,H^2}{2}\left(1 + \frac{1}{k^2}\right) \ldots (2)$$

Third.—Pressure on base, reservoir full: (From the conditions imposed, maximum at the toe, 0 at the heel).

Take moments about the center of gravity of the base:

$$\text{Moment of horizontal pressure} = \frac{w\,H^2}{2} \times \frac{H}{3} = \frac{w\,H^3}{6}$$

$$\text{Moment of upward pressure} = \frac{w\,H\,B}{2} \times \frac{B}{6} = \frac{w\,H\,B^2}{12}$$

Mr. Coburn.

Moment of weight of lower triangle $= \frac{2.3\,w\,H\,B}{2} \times \frac{B}{6} = \frac{2.3\,w\,H\,B^2}{12}$

Moment of weight of upper triangle $= \frac{2.3\,w\,h\,b}{2} \times \left(\frac{B}{2} - \frac{2\,b}{3}\right) = \frac{2.3\,w\,H\,B^2}{12}\left(\frac{3\,k-4}{k^3}\right)$

Reaction $= P_{av.}\ B \times \frac{B}{6} = \frac{P_{av.}\ B^2}{6}$

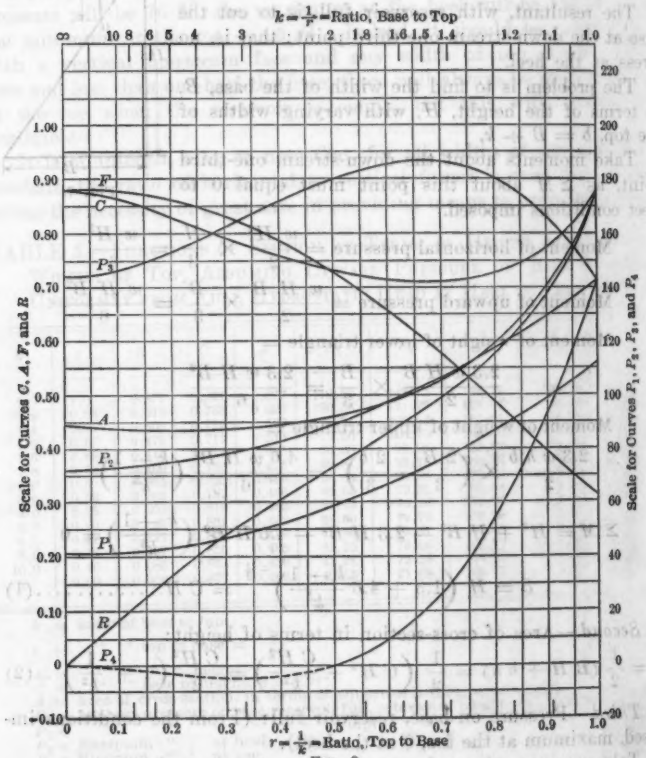

FIG. 8.

Equate, multiply by 6, and divide by B^2, and we get

$$P_{av.} = w\,H\left[\frac{1}{C^2} + \frac{1}{2} - 1.15\left(1 + \frac{3\,k-4}{k^3}\right)\right] = P_1\,H \ldots . (3)$$

$P_{max.}$ = twice $P_{av.}$

Mr. Coburn.

Fourth.—Pressure on base, reservoir empty:
Take moments about the center of gravity of the base.

Moment of weight of lower triangle =

$$\frac{2.3\,w\,H\,B}{2} \times \frac{B}{6} = \frac{2.3\,w\,H\,B^2}{12}$$

Moment of weight of upper triangle =

$$\frac{2.3\,w\,H\,B}{2\,k^2}\left(\frac{B}{2} - \frac{2\,B}{3\,k}\right) = \frac{2.3\,w\,H\,B^2}{12}\left(\frac{3\,k-4}{k^3}\right)$$

$$\Sigma\,W = \frac{2.3\,w\,H\,B}{2}\left(1 + \frac{1}{k^2}\right); \quad \Sigma\,M = \frac{2.3\,w\,H\,B^2}{12}\left(1 + \frac{3\,k-4}{k^3}\right).$$

$$\text{Moment of arm of resultant} = \frac{\Sigma\,M}{\Sigma\,W} = \frac{B}{6}\left(\frac{k^3+3\,k-4}{k^3+k}\right)$$

or
$$e = \frac{C\,H}{6}\left(\frac{k^3+3\,k-4}{k^3+k}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Now, the average pressure on the base $= \Sigma\,W \div B$

$$= P_2\,H = \frac{2.3\,w\,H}{2}\left(1 + \frac{1}{k^3}\right) = 1.15\,w\,H\left(1 + \frac{1}{k^3}\right) \ldots (5)$$

$$P_3 = (P_{max.}) = P_2\left(1 + \frac{6\,e}{B}\right) = 1.15\,w\,H\left(1 + \frac{1}{k^3}\right)\left(1 + \frac{k^3+3\,k-4}{k^3+k}\right)$$

$$= 2.3\,w\,H\left(\frac{k^3+3\,k-2}{k^3}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

$$P_4 = (P_{min.}) = P_2\left(1 - \frac{6\,e}{B}\right) = 1.15\,w\,H\left(1 - \frac{1}{k^3}\right)\left(1 + \frac{k^3+3\,k-4}{k^3+k}\right)$$

$$= 2.3\,w\,H\,\frac{2-k}{k} \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

Fifth.—Ratio of horizontal to vertical forces:

$$F = \Sigma\,H \div \Sigma\,V$$

$$= \frac{w\,H^2}{2} \div \left(\frac{2.3\,w\,H\,B}{2} + \frac{2.3\,w\,H\,B}{2\,k^2} - \frac{w\,H\,B}{2}\right)$$

$$= \frac{H}{B}\left(\frac{k^2}{1.3\,k^2 + 2.3}\right) = \frac{1}{C}\,\frac{k^2}{1.3\,k^2 + 2.3} \quad \ldots\ldots\ldots\ldots (8)$$

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Paper No. 1362

THE CHERRY STREET BRIDGE, TOLEDO, OHIO*

By Clement E. Chase, Jun. Am. Soc. C. E.

With Discussion by Messrs. Edward Godfrey, James Ritchie, and Clement E. Chase.

Synopsis.

This paper describes the general design and the most interesting and instructive features of the construction of a massive reinforced concrete arch bridge across the Maumee River at Toledo, Ohio.

After a short résumé of the history of the former bridge at this site and of the new bridge project, the loadings and general features of the design are stated. The construction work is followed in some detail from its beginning in 1910 to the opening for use of the first half of the structure during the winter of 1912-13. The construction of the second half of the bridge, being to all intents a duplication of the work on the first half, is merely outlined up to January 1st, 1914, at which time the structure was practically complete.

The description of construction deals with the following features: The building of the bridge in parallel halves, carrying traffic on the old structure at the same time, until the first half was completed; the use of deep, single-wall coffer-dams and attempts to dig wells to rock from the bottom of the pumped-out dams by the Chicago method; change of plans by reason of the proven hazard of this first plan, and the utilization of steel cylinders sunk inside the unwatered dams; the

* Presented at the meeting of December 1st, 1915.

filling of these cylinders with concrete and the sealing of the coffer-dams by concrete deposited under water; the construction plant and methods of depositing concrete under water, with a statement of the general principles; the collapse of a steel coffer-dam, through failure of the bracing, and its repair and re-use; the use of movable steel centers to support the arch concrete, and the expedients resorted to for increasing the width (of the arch barrel) poured at one time; the forms, plant, and methods of work in building the arches, spandrel walls, and viaduct; the procedure followed in water-proofing and back-filling; and the erection of the bascule span. Special attention is paid to the esthetic appearance of the bridge. The paper closes with a summary of unit prices, quantities, and amounts expended for the different parts of the work.

Historical.

The main portion of the City of Toledo, Ohio, is separated from an important industrial and residential section, known as East Toledo, by the Maumee River, across which, from the earliest history of the community, the Cherry Street Bridge has been the main thoroughfare. Following several predecessors—of types corresponding to the progress of the art of bridge building in the United States—the iron structure, which has been recently replaced, served to carry the growing traffic across the river from 1883 until January, 1913. Several years before that time, it was realized that the bridge was inadequate for the constantly increasing loads which it had to carry, and that, with the larger number of lake carriers docking above its site, a more rapid form of movable span was needed than the existing swing span. The question of type and location for the new bridge was debated heatedly, at great length, and with such intense partisanship that the whole project was held up for a long period. The main points at issue were finally settled—the new structure should be of concrete, with a double-leaf lift-span across the channel, and should cross the river, as before, at Cherry Street.

Design.

As finally located, the south half of the new and much wider structure covers the site of the old one. For a time it was planned to construct temporary piers alongside the old ones and shift the

spans sideways so as to clear the bridge site. Later, this plan was abandoned, largely on account of anticipated danger from ice, and it was decided to build the new bridge in longitudinal halves. The north portion was to be completed first, the old bridge was then to be removed, and finally the arches were to be extended to their full width of 73 ft. 10 in. The width of Cherry Street is maintained practically unchanged as it passes over the bridge, the roadway being 52 ft. between curbs and the sidewalks 9 ft. wide. Two car tracks are provided in the center of the roadway.

There is a clear channel opening of 200 ft., closed by a double-leaf bascule span. Approaching this, the roadway has a grade of 2.02% from the east and 1.35% from the west, which gives enough elevation at the bascule to allow a clearance of 29 ft. under the steel, with span closed. This is quite enough to pass the tugs and "sandsuckers" which caused a considerable percentage of the openings of the old draw-span. Maintaining river traffic through both bridges during the construction of the new one, required the layout of the new channel as shown on Fig. 1, so that in order to open it, the old draw might be swung one way until the new bridge should go into commission. Fig. 1 also shows the layout and principal dimensions of the arch openings. To the east of the channel there are five spans, decreasing in length from 108 ft. to 66 ft. 6 in. and in rise from 28 ft. 6 in. to 16 ft., in order to conform to the grade. To the west there are two more of the 108-ft. arches—the grade here being taken care of by dropping both ends of the arch nearest the shore by 1½ ft. These 108-ft. arches have a thickness of 7 ft. at the springing line and 2 ft. 3 in. at the crown. Their intrados is an ellipse with a semi-major axis of 54 ft. and a semi-minor axis of 29 ft. Fig. 2 gives the dimensions of the various arch rings. The assumptions of load used in the design include a sand fill weighing 100 lb. per cu. ft.; 50-ton interurban cars in trains on both tracks, each covering a width of 10 ft.; and a live load of 150 lb. per sq. ft. on the remainder of the roadway and of 100 lb. per sq. ft. on the sidewalks. A maximum stress in the concrete of 450 lb. per sq. in. was specified, and a temperature range of 70° was assumed. The reinforcement consists of latticed ribs, 2 ft. 6 in. from center to center, and transverse rods, along the extrados and intrados, 3 ft. apart.

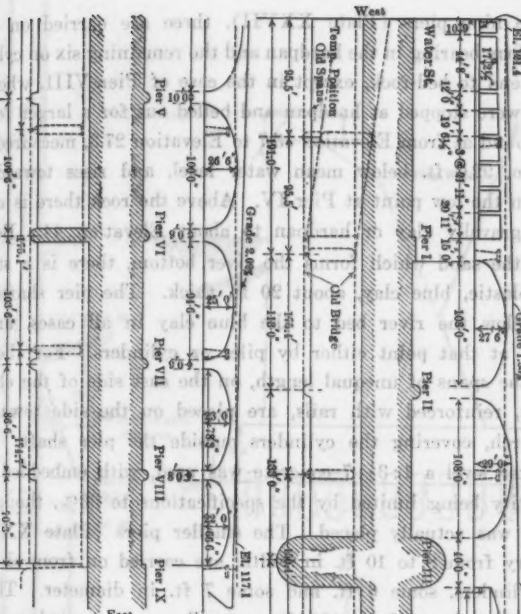
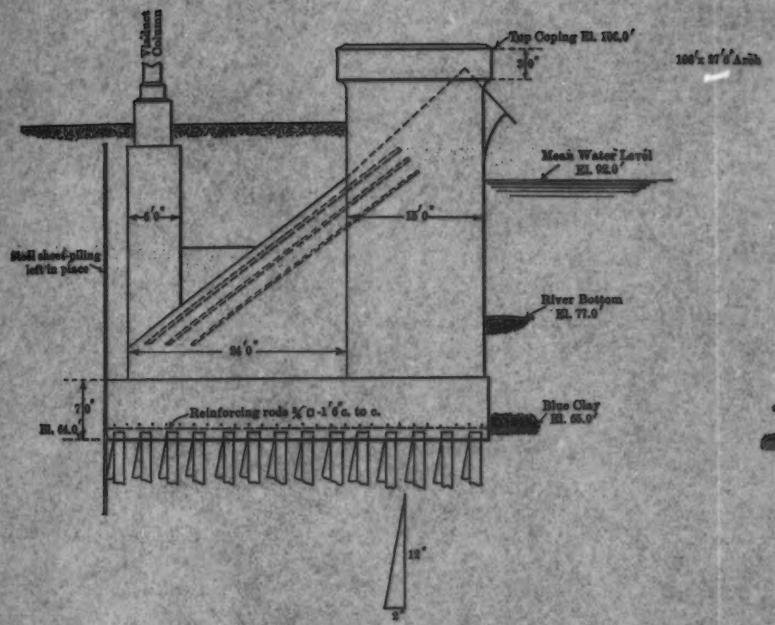


FIG. 1.

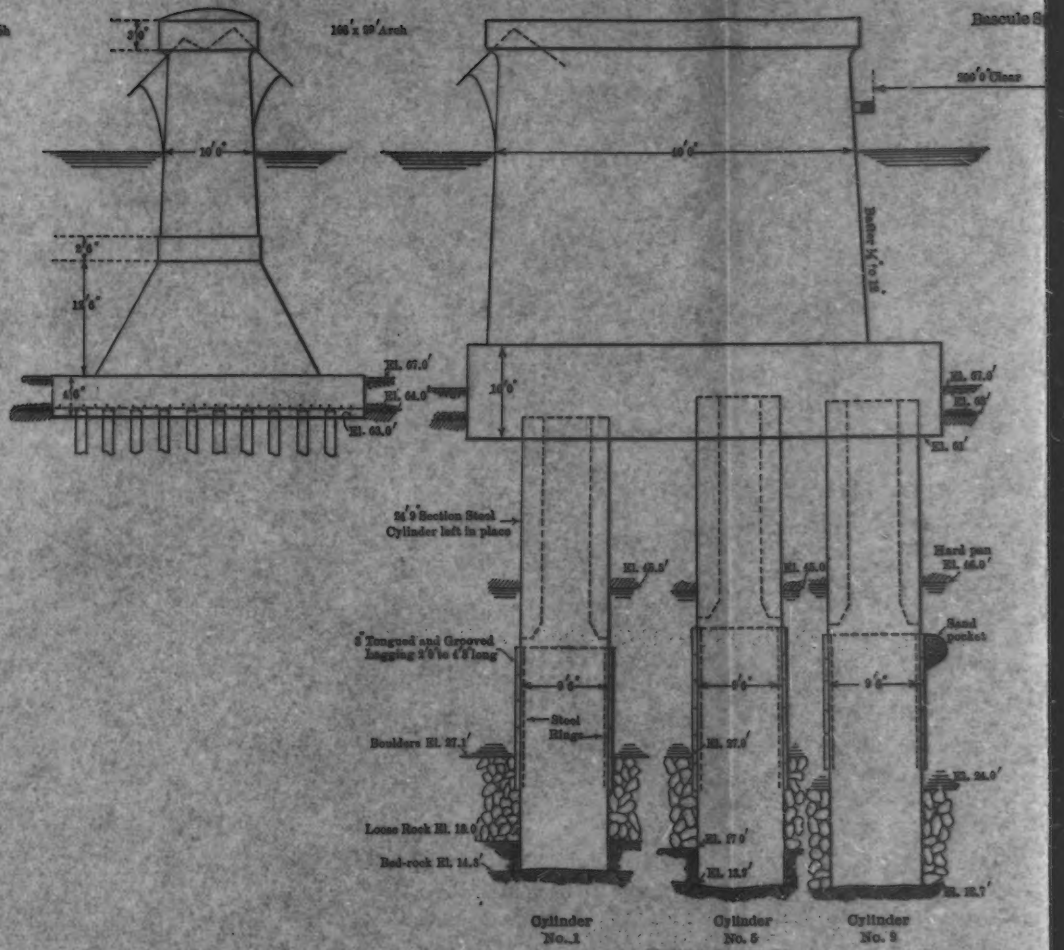
CHERRY STREET BRIDGE, TOLEDO, OHIO.
GENERAL PLAN OF OLD AND NEW STRUCTURES.

Of the nine piers (Plate XXVII), three are carried on piling driven to firm bearing in the hardpan and the remaining six on cylinders which extend to bed-rock, except in the case of Pier VIII, where the cylinders were stopped at hardpan and belled out for a larger footing. Rock is found at from Elevation 13.7 to Elevation 27.3, measured from the datum, 92.0 ft. below mean water level, and rises toward each shore from the low point at Pier IV. Above the rock there is a layer of hard, gravelly clay or hardpan to about Elevation 44. Between this and the sand which forms the river bottom, there is a stratum of soft, plastic, blue clay, about 20 ft. thick. The pier shafts were carried below the river bed to the blue clay in all cases, and are supported at that point either by piles or cylinders. For the piers between the spans of unequal length, on the east side of the channel, buttresses, reinforced with rails, are placed on the side toward the smaller arch, covering the cylinders outside the pier shaft. Below mean water level a 1:3½:7 concrete was used, with embedded stone, the quantity being limited by the specifications to 33%, though less than this was actually placed. The smaller piers (Plate XXVIII), which vary from 8 to 10 ft. in width, are carried on from eleven to twelve cylinders, some 6 ft. and some 7 ft. in diameter. The two large bascule piers, 40 by 126 ft. over all, are supported on eleven cylinders, 9 ft. 6 in. in diameter. The two abutment piers, I and IX, are carried on piles, 3 ft. from center to center and 25% of these piles are driven to a 2 in 12 batter. Pier II, which has a 108-ft. arch on each side, is carried on 370 piles, supporting a 24 by 105-ft. footing.

The construction of the bridge in longitudinal halves necessitated a central spandrel wall to retain the fill in the north half until the completion of the whole bridge. For this, a reinforced concrete wall, 18 in. thick, with counterforts 9 ft. from center to center, was designed. The outside spandrel wall was shown on the original plans —on which the contractor's bid was based—as a gravity section. and was built with a base forty-two one-hundredths of the height and a fixed top width of 1 ft. 6 in. The back surface of the wall is warped, as the batter changes constantly, but this caused very little difficulty in forming. A tile conduit to carry power and lighting cables across the bridge rests on a reinforced concrete slab supported under the sidewalks by brackets built into the outer spandrel wall.



ABUTMENT, PIER I.

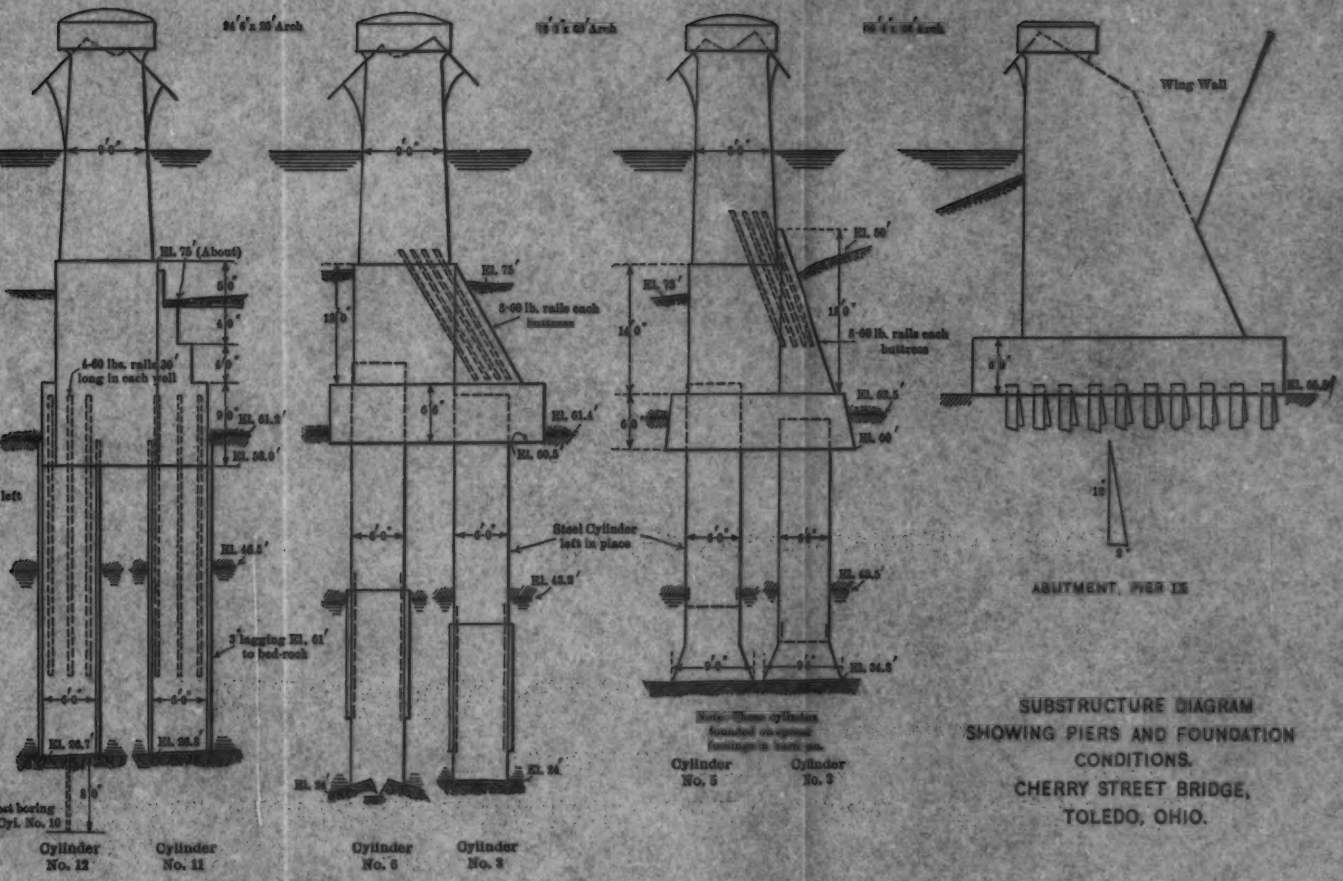


PIER II

BASCULE PIER III

Bascule Span
200'0" Clear
40'0"
Batter ¼" to 1ft
El. 67.0'
El. 62'
El. 61'
El. 48.5'
El. 48.0'
Hard pan
El. 46.0'
Sand pocket
9'0"
El. 27.0'
El. 24.0'
El. 17.0'
El. 13.9'
El. 13.7'
Cylinder No. 5
Cylinder No. 9
BASCULE, PIER III
40'0"
11'0"
El. 64'
El. 61.2'
El. 61'
Two 6'0" sections steel cylinder left in place
El. 48.0'
El. 42.2'
9'6"
Loose Rock
El. 14.9'
El. 13.7'
El. 19.5'
El. 15.0'
El. 17.5'
El. 15.5'
Cylinder No. 11
Cylinder No. 8
Cylinder No. 4
BASCULE, PIER IV
105'x 33'6 Arch
3'0"
100'x 26'6 Arch
10'0"
El. 80' (About)
El. 75' (About)
8-60 lb. 18 ft. long rails each buttress
16'0"
7'2"
El. 61.2
El. 56.2
Steel Cylinder left in place
El. 44.6'
El. 45.0'
6'0"
Rock
El. 28.5'
El. 27.2'
Cylinder No. 6
Cylinder No. 3
Test borin in Cyl. No.
PIER V

PLATE XXVII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXX, No. 1367.
CHASE ON
CHERRY STREET BRIDGE, TOLEDO, OHIO.



PIER VI PIER VII PIER VIII

SUBSTRUCTURE DIAGRAM
SHOWING PIERS AND FOUNDATION
CONDITIONS.
CHERRY STREET BRIDGE,
TOLEDO, OHIO.

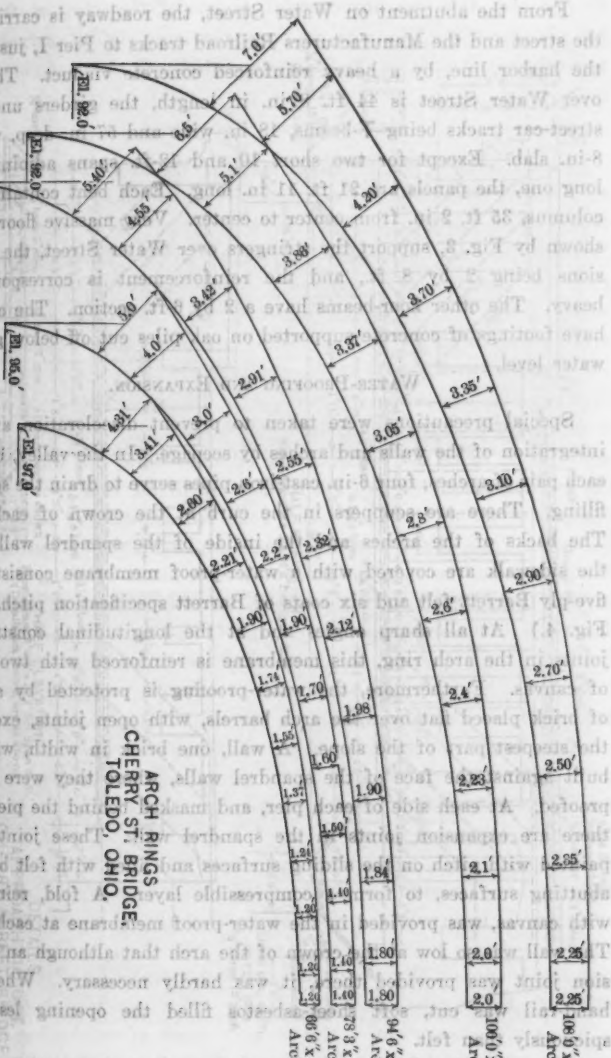


FIG. 2.—ARCH RINGS CHERRY ST. BRIDGE, TOLEDO, OHIO.

From the abutment on Water Street, the roadway is carried over the street and the Manufacturers Railroad tracks to Pier I, just inside the harbor line, by a heavy reinforced concrete viaduct. The span over Water Street is 44 ft. 8 in. in length, the girders under the street-car tracks being T-beams, 18 in. wide and 57 in. deep, with an 8-in. slab. Except for two short 10 and 12-ft. spans adjoining the long one, the panels are 21 ft. 11 in. long. Each bent contains three columns, 35 ft. 2 in. from center to center. Very massive floor-beams, shown by Fig. 3, support the stringers over Water Street, the dimensions being 2 by 8 ft., and the reinforcement is correspondingly heavy. The other floor-beams have a 2 by 6-ft. section. The columns have footings of concrete supported on oak piles cut off below ground-water level.

Water-Proofing and Expansion.

Special precautions were taken to prevent discoloration and disintegration of the walls and arches by seepage. In the valley, between each pair of arches, four 6-in. cast-iron pipes serve to drain the spandrel filling. There are scuppers in the curb at the crown of each arch. The backs of the arches and the inside of the spandrel wall up to the sidewalk are covered with a water-proof membrane consisting of five-ply Barrett felt and six coats of Barrett specification pitch. (See Fig. 4.) At all sharp angles and at the longitudinal construction joints in the arch ring, this membrane is reinforced with two layers of canvas. Furthermore, the water-proofing is protected by a layer of brick placed flat over the arch barrels, with open joints, except in the steepest part of the slope. A wall, one brick in width, was also built against the face of the spandrel walls, where they were water-proofed. At each side of each pier, and masked behind the pier face, there are expansion joints in the spandrel wall. These joints were painted with pitch on the sliding surfaces and filled with felt between abutting surfaces, to form a compressible layer. A fold, reinforced with canvas, was provided in the water-proof membrane at each joint. The wall was so low at the crown of the arch that although an expansion joint was provided there, it was hardly necessary. Where the hand-rail was cut, soft sheet-asbestos filled the opening less conspicuously than felt.

The spandrel walls were reinforced throughout their middle-thirds, in order to render the formation of cracks at that point less likely.

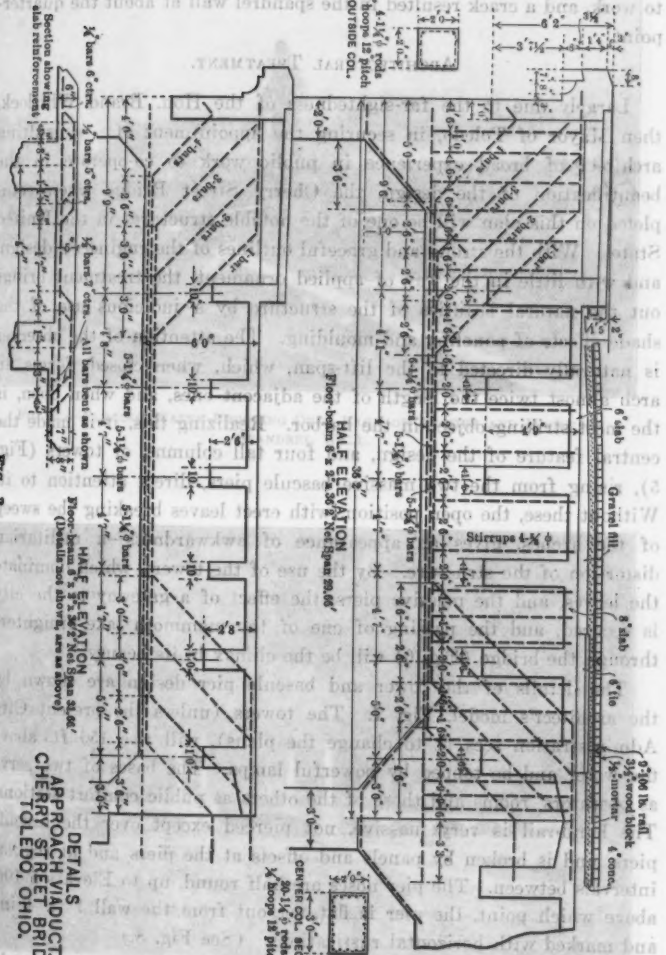

DETAILS
APPROACH VIADUCT.
CHERRY STREET BRIDGE
TOLEDO, OHIO.
HALF ELEVATION
Floor-beam 6'x 2'-3" x 36'-2" Net Span 29.66'
HALF ELEVATION
Floor-beam 6'x 2'-3" x 36'-2" Net Span 31.66'
(Details not shown, are as above)
Section showing slab reinforcement
All bars bent as shown
Stirrups 4-¾" ϕ
Gravel fill
6" slab
6" tie
6" conc.
½" mortar
3½" wood block
9"x10 lbs. rail
CENTER COL. SECT.
OUTSIDE COL.

FIG. 3.

In one case, this was unsuccessful, as one of the joints at the main bascule pier (Pier IV), with an unusually large sliding surface, failed to work, and a crack resulted in the spandrel wall at about the quarter-point.

Architectural Treatment.

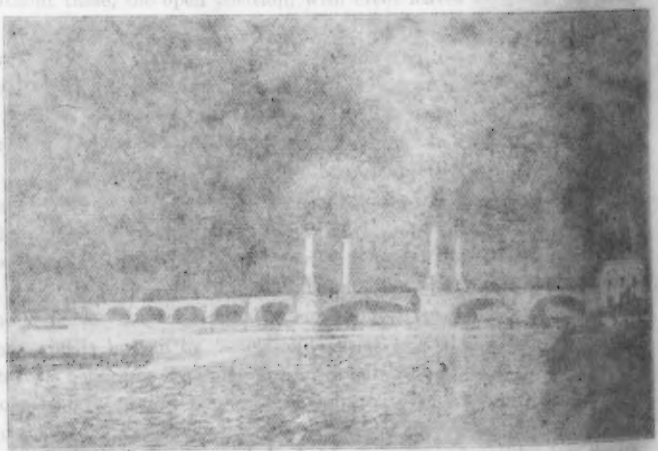
Largely due to the far-sightedness of the Hon. Brand Whitlock, then Mayor of Toledo, in securing the appointment of a consulting architect of broad experience in public work to co-operate in the beautification of the design, the Cherry Street Bridge when completed on this plan will be one of the notable structures in the United States. With the strong and graceful outlines of the engineer's design, and with little in the way of applied ornament, the treatment brings out the natural beauties of the structure by a judicious use of the shade effects of paneling and moulding. The attention of the observer is naturally directed to the lift-span, which, when closed, forms an arch almost twice the length of the adjacent ones, and when open, is the most striking object in the harbor. Realizing this, it is made the central feature of the design, and four tall columns, or towers (Fig. 5), rising from the two massive bascule piers, direct attention to it. Without these, the open position, with erect leaves breaking the sweep of the arches, gives an appearance of awkwardness—a utilitarian distortion of the structure. By the use of the towers, which dominate the leaves, and the massive piers, the effect of a gateway to the city is secured, and the passing of one of the mammoth lake freighters through the bridge (Fig. 6) will be the climax in its beauty.

The details of the tower and bascule pier design are shown by the architect's model, Fig. 7. The towers (unless the present City Administration sees fit to change the plans), will rise 150 ft. above the water and be topped by powerful lamps. The bases of two serve as operators' rooms and those of the others as public comfort stations. The hand-rail is very massive, not pierced except over the bascule piers, and is broken by panels and offsets at the piers and at regular intervals between. The pier noses are half round, up to Elevation 106, above which point, the pier is flat, set out from the wall 1 ft. 4 in., and marked with horizontal rustications. (See Fig. 8.)

The surface treatment of the bridge consists of roughening certain portions, exposing the special aggregate of small gray and yellow limestone chips which was cast against the forms, and, for contrast,

FIG. 4.—PLACING WATER-PROOFING OVER BACK OF ARCHES AND INSIDE OF SPANDREL WALL.

FIG. 5.—ARCHITECT'S DRAWING OF COMPLETED CHERRY STREET BRIDGE.



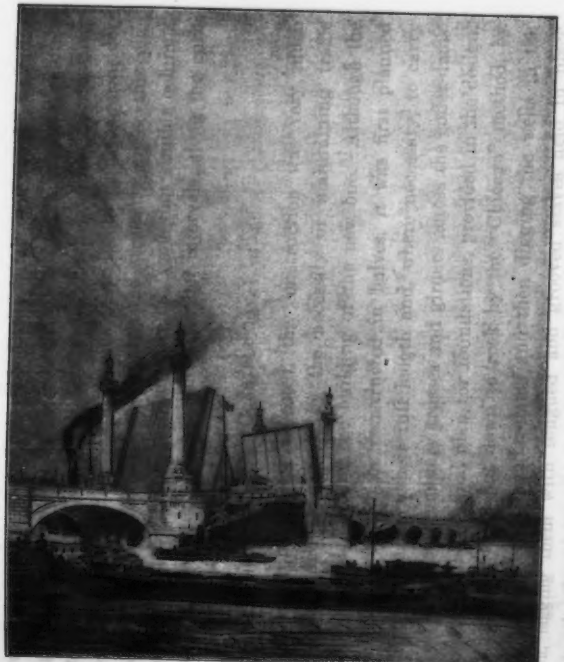


FIG. 6.—ARCHITECT'S DRAWING: FREIGHTER PASSING THROUGH OPEN BASCULE.

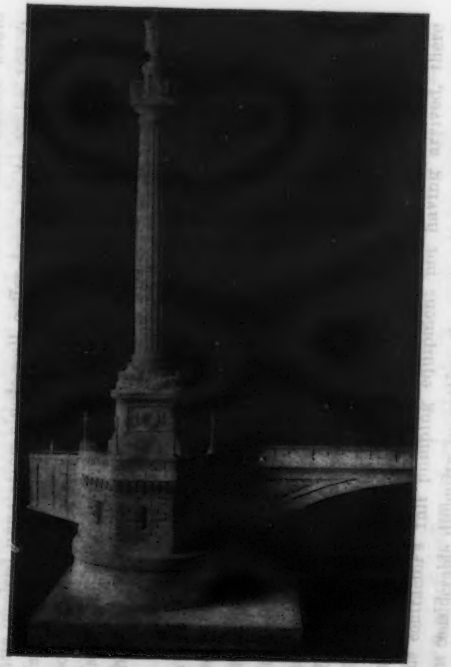


FIG. 7.—ARCHITECT'S MODEL OF BASCULE PIER AND COLUMN.

leaving untreated or smooth-rubbed, the bands and mouldings. The lighting system first contemplated consisted of a line of metal columns at close intervals along each hand-rail. Each column was to carry a single large globe containing a cluster of lights. Further consideration of the probable cost of lighting the structure under this plan forced a change. The system adopted has somewhat similar columns, but, with less powerful lights, and at longer intervals, along the curb.

Construction.

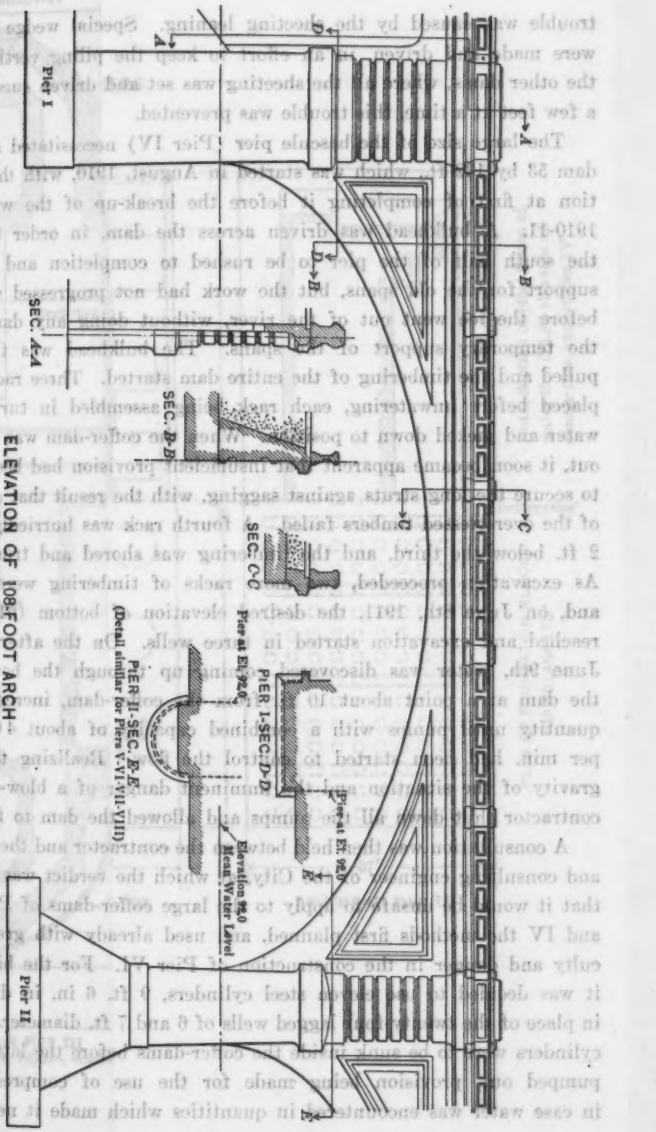
As might have been expected, the construction was very much complicated and lengthened by the necessity of maintaining traffic on the old bridge during the building of the new one. Although the superstructure had to be constructed in halves, it was first planned to build all the piers their full length and, where necessary, to carry the old bridge on temporary trusses and girders across the coffer-dams. According to the original plan for foundations, provided in the design, cylinders were to be put down to rock by the "Chicago" method, by first pumping out the coffer-dams and then digging the wells in the open, lagging them with tongued and grooved staves held in place by iron bands, as the excavation proceeded.

This daring and unusual procedure in work of this nature would have involved unwatering single-wall coffer-dams up to 53 by 152 ft. in size against a head of 30 ft. of water, and then starting from the bottom thus exposed and digging from 12 to 24 wells as much as 45 ft. farther, to rock. The work would have been through a blue clay, shown by the borings to contain pockets and streaks of sand.

Work was begun under this plan on Pier VI, on August 9th, 1910. A single-wall coffer-dam, 24 by 90 ft., of steel sheet-piling 46 ft. in length, was driven around the pier site. For that part of the dam under the old bridge, the sheeting was assembled complete for each side and floated in, suspended in a vertical position from staging built on a barge. To drive it, a heavy forging was hung from the top of the pile, which allowed the hammer to work several feet below the pile top. The timbering was placed as the dam was unwatered, and was very heavy, consisting of seven racks, 4 ft. 3 in. apart vertically. The contractor's full pumping equipment not having arrived, there was considerable difficulty in getting the water "started down" in this dam. Three sand dredges, equipped with two 8-in. and one 10-in.

centrifugal pumps, were finally pressed into service to aid the 8-in., steam-driven centrifugal and two reciprocating pumps of the contractor's plant. As soon as a difference of head was secured, a mixture of manure and ashes, dropped along the dam on the outside, successfully stopped the leaks. Grouting through holes in the sheeting was also resorted to where the leaks occurred below the level of the river bed on the outside. After the excavation and timbering were completed, the twelve wells were started in the bottom of the coffer-dam and successfully sunk to rock, the work being done in a constant shower of water in many cases and despite the hazards of the method, which were fully realized by the contractor's men. Continual pumping was required in several of the holes, and emphasized the danger of a blow-out. At one time, it seemed as though it would be necessary to excavate a couple of the wells through water, have a diver clean off the rock, and then deposit concrete under water. The leakage was finally controlled by grouting through pipes driven down alongside the cylinders on the outside and through holes drilled through the lagging. As soon as they were completed, the wells were concreted to a point 1 ft. above the bottom of the pier excavation. Six 60-lb. rails were placed in the upper portion of the cylinder as reinforcement. The pier shaft was carried up in successive lifts to the under side of each rack of timbering. After waiting 48 hours in each case, the dam was braced against the concrete, the rack removed, and the concreting continued. In subsequent dams, where much less timbering was used, the struts were concreted into the pier to within 10 ft. of normal water level, Elevation 92, and were afterward cut off flush with the face of the pier.

At about the same time, work was begun on Piers IV and IX, at both of which temporary supports had to be provided for the old spans, to allow the piers to be removed. At Pier IX two plate-girder spans, and at Pier IV four timber Howe trusses, were utilized to support the spans over the center of the coffer-dams. Great difficulty was experienced in driving the dam at Pier IX, the steel piling being deflected, buckled, and opened by the obstacles encountered. Work on this pier, though begun at this early date, was not finished until May 9th, 1912. Ultimately, an outer wall of Wakefield sheeting had to be driven around the worst portion of the dam. Under the bridge, where it was necessary to drive the sheeting down through the floor, one pile at a time,



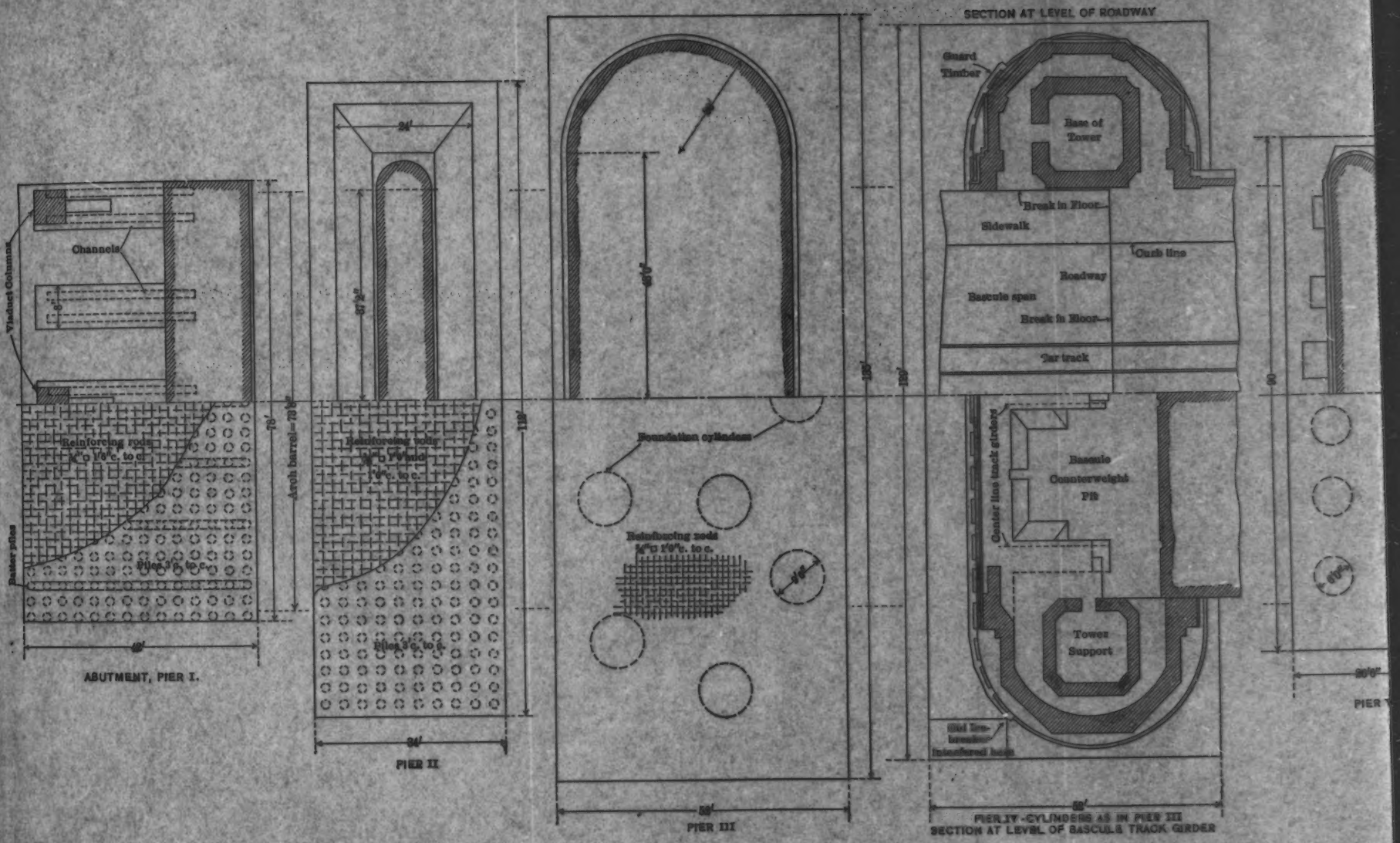
ELEVATION OF 108-FOOT ARCH
PIER I TO PIER II
FIG. 8.

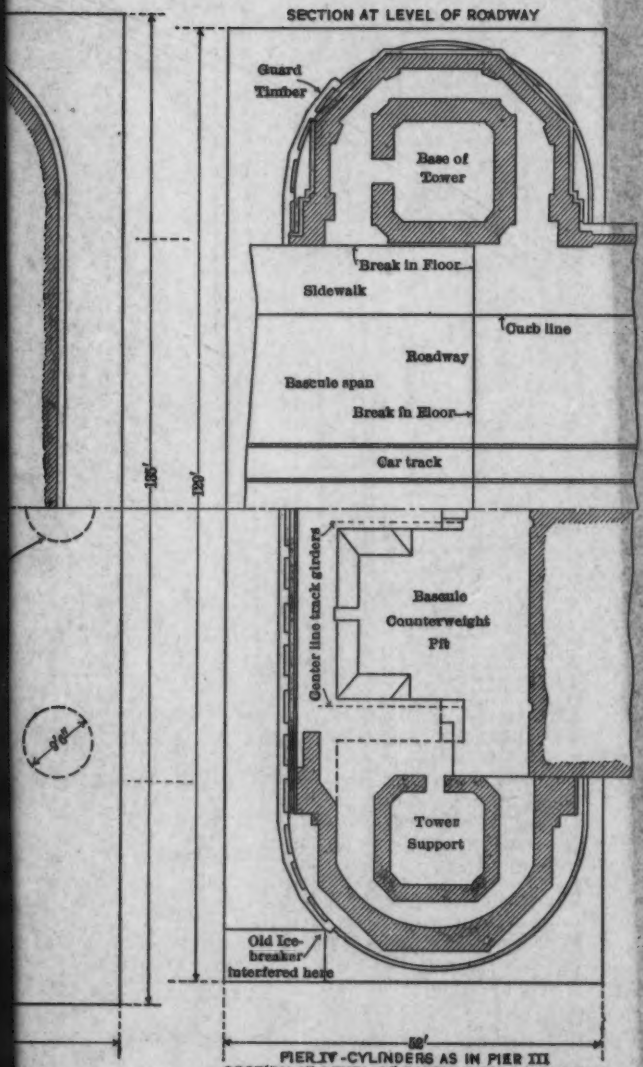
PIER II-SEC. E-E
(Detail similar for Piers V-VI-VII-VIII)

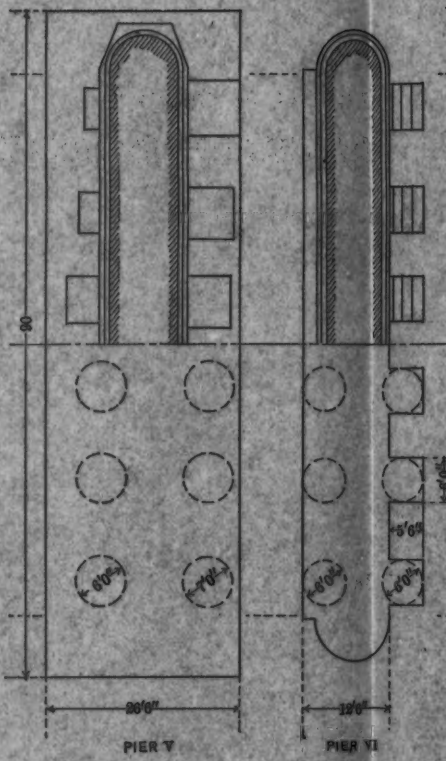
trouble was caused by the sheeting leaning. Special wedge sections were made and driven, in an effort to keep the piling vertical. In the other dams, where all the sheeting was set and driven successively a few feet at a time, this trouble was prevented.

The large size of the bascule pier (Pier IV) necessitated a coffer-dam 53 by 132 ft., which was started in August, 1910, with the intention at first of completing it before the break-up of the winter of 1910-11. A bulkhead was driven across the dam, in order to allow the south half of the pier to be rushed to completion and furnish support for the old spans, but the work had not progressed very far before the ice went out of the river, without doing any damage to the temporary support of the spans. The bulkhead was therefore pulled and the timbering of the entire dam started. Three racks were placed before unwatering, each rack being assembled in turn above water and jacked down to position. When the coffer-dam was pumped out, it soon became apparent that insufficient provision had been made to secure the long struts against sagging, with the result that a couple of the overstressed timbers failed. A fourth rack was hurriedly placed 2 ft. below the third, and the timbering was shored and trussed up. As excavation proceeded, two more racks of timbering were placed and, on June 8th, 1911, the desired elevation of bottom (61.0) was reached and excavation started in three wells. On the afternoon of June 9th, water was discovered coming up through the bottom of the dam at a point about 10 ft. from the coffer-dam, increasing in quantity until pumps with a combined capacity of about 4 000 gal. per min. had been started to control the flow. Realizing then the gravity of the situation and the imminent danger of a blow-out, the contractor shut down all the pumps and allowed the dam to flood.

A consultation was then held between the contractor and the officials and consulting engineer of the City, at which the verdict was reached that it would be unsafe to apply to the large coffer-dams of Piers III and IV the methods first planned, and used already with great difficulty and danger in the construction of Pier VI. For the big piers, it was decided to use eleven steel cylinders, 9 ft. 6 in. in diameter, in place of the twenty-four lagged wells of 6 and 7 ft. diameter. These cylinders were to be sunk inside the coffer-dams before the latter were pumped out, provision being made for the use of compressed air in case water was encountered in quantities which made it necessary.


SECTION AT LEVEL OF ROADWAY
Guard Timber
Base of Tower
Break in Floor
Sidewalk
Curb line
Roadway
Bascule span
Break in floor
2ar track
Channels
Viaduct Column
Reinforcing rods
Arch barrel = 70'8"
Foundation cylinders
Reinforcing rods
Bascule Counterweight Pit
Center line track girders
Tower Support
Steel Inter-braces
Reinforced beam
ABUTMENT, PIER I.
PIER II
PIER III
PIER IV - CYLINDERS AS IN PIER III
SECTION AT LEVEL OF BASCULE TRACK GIRDER

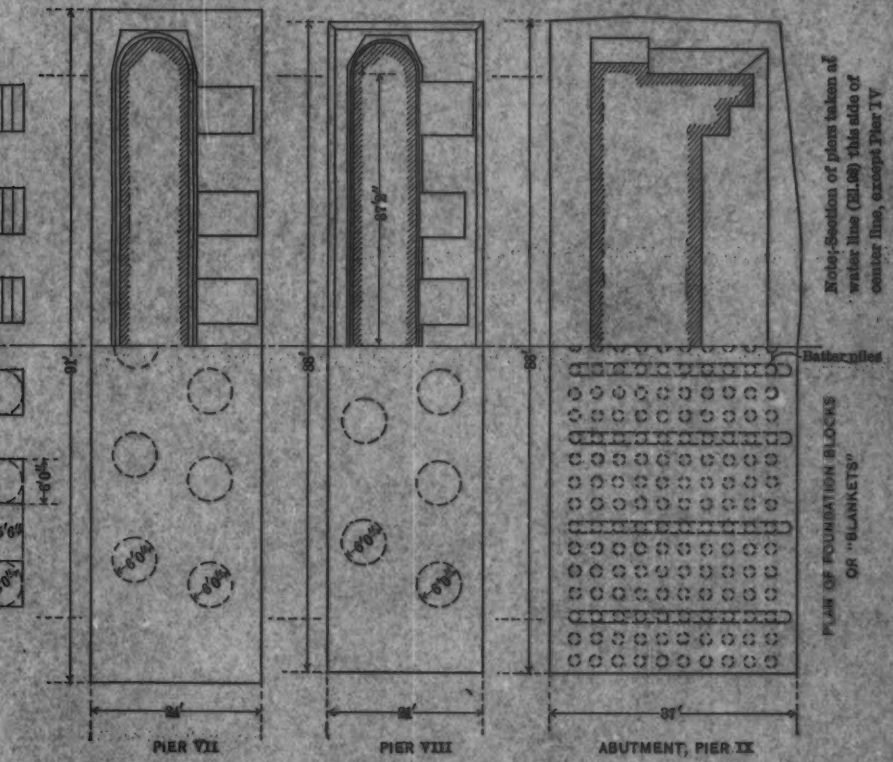

SECTION AT LEVEL OF ROADWAY
Guard Timber
Base of Tower
Break in Floor
Sidewalk
Curb line
Roadway
Bascule span
Break in Floor
Car track
Center line track girders
Bascule Counterweight Pit
Tower Support
Old Ice-breaker interfered here
135'
129'
92'
PIER IV - CYLINDERS AS IN PIER III
SECTION AT LEVEL OF BASCULE TRACK GIRDER


90
20'0"
12'0"
PIER V
PIER VI

SUBSTRU

PLATE XXVIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXX, No. 1382.
CHASE ON
CHERRY STREET BRIDGE, TOLEDO, OHIO.

STRUCTURE DIAGRAM, SHOWING PLAN OF PIERS
CHERRY STREET BRIDGE
TOLEDO, OHIO



Below the elevation at which the cylinders were to be stopped the wells were to be carried down to rock with timber lagging, as in the original plan. After rock had been reached, the cylinders were to be filled with concrete up to the elevation of the base of the pier, and then the section of steel cylinder from there to the water's surface was to be removed. A layer or "blanket" of concrete, 10 ft. thick, was then to be placed over the entire area of the coffer-dam, being deposited under water by suitable means. The dam would then be timbered, pumped, and the pier shaft completed. Piers V, VII, and VIII were to be founded according to the original design, but, in order to expedite the work and prevent accidents, steel cylinders were substituted for lagging. To offset the increased cost of the foundations, piles were substituted for cylinders under Piers I, II, and IX, Pier II having little unbalanced thrust and the abutment piers affording an opportunity for a large spread of footing and massive piers.

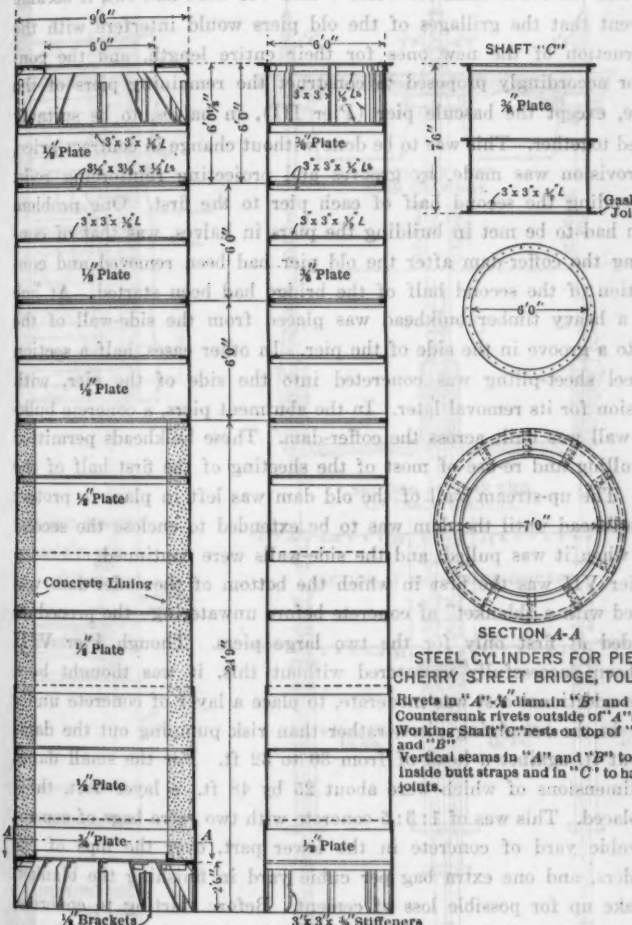
The steel cylinders for Pier IV, as called for under these plans (Fig. 9), had a cutting-edge section 24 ft. 9 in. in length, to which could be bolted additional sections 6 ft. in length. A shelf around the inside of the bottom section near the cutting-edge supported the lining of concrete, 1 ft. 3 in. in thickness, which furnished part of the weight required for sinking. A wooden bulkhead was placed in the cylinders to float them while the lining was being placed, and was removed after the weight of the concrete had caused the cutting-edge to seal itself in the clay. As the cylinder followed the excavation down, additional top sections 6 ft. in length were added as needed. The third or fourth 6-ft. section added was a reducing section, decreasing the diameter from 9 ft. 6 in. to 6 ft., and allowed the placing of concrete weight blocks around the 6-ft. cylinder to get the needed sinking weight. The blue clay was cut with spades and removed in small tilting buckets. One stiff-leg derrick, mounted alongside the coffer-dam on piles, served all the holes except those under the bridge. The spoil from these latter was hoisted by runner lines working through snatch-blocks hung from the Howe trusses.

At Elevation 43.5 the material changed to a very hard dry clay or hardpan, which it was necessary to break up with a mattox or split off with bars and wedges. Limestone boulders, of all sizes up to ½ cu. yd., were encountered in this stratum, with occasional sand pockets, only a few of which showed any traces of water. The holes in most

cases were dry, trouble being usually caused by water following down alongside the cylinder after a sudden drop. At first, the steel cylinders were sunk to their limit before continuing them down to rock with wooden lagging. Later, after a Court decision that only the 24 ft. 9-in. section should be paid for, weight was taken off the cylinders as soon as all but a foot or so of this section was embedded in the clay. In continuing the wells below the cutting-edge, the shaft was excavated for distances of from 2 to 5 ft., and the hole was then lagged with short sections of double-grooved yellow pine sheeting, with radial edges and oak tongues. Steel hoops, 9 ft. 6 in. in diameter, made in three sections which bolted together, braced the lagging in place. In the last few feet to the bottom, where the material was favorable, the walls were often left unlagged. Bed-rock was found at elevations varying from 13.7 to 18.2 at this pier, and was usually a polished, flat limestone. The eleven cylinders of Pier IV were sunk a total distance of 490 ft. from July 28th to November 15th, 1911, with an average of 23½ days to each cylinder. The average daily progress, when work was not unduly delayed, was 2.7 ft.

During the next season, in putting down the wells for Pier III, several changes in the method of prosecuting the work resulted in shortening the time materially. An average interval of 17½ days elapsed on this side of the channel for the excavation of each cylinder, the work going forward at the rate of 3.1 ft. for each of the 15½ actual working days. In the first place, the sheeting was driven on only three sides of the dam, so that the cylinders could be concreted until nearly aground and then floated into position, instead of being lifted over the dam as before. The thickness of the shell of concrete was increased to 2 ft., leaving a shaft of 5 ft. 5 in., and making less additional sinking weight necessary in the shape of concrete blocks. Two derricks were provided for hoisting the spoil. Gangs of three men in each hole worked in three shifts of 8 hours each.

Concrete for the cylinders was mixed on the floating plant and chuted to a hopper, from which it was dropped from 35 to 78 ft. through a 10-in. pipe to place. Observation of the concrete in the cylinders, during stoppages in the work, showed practically no separation of materials and a very marked packing and tamping of the concrete. Water at times accumulated in leaky holes, or from over-wet batches, to such an extent that drying the mix did not take it up, and



STEEL CYLINDERS FOR PIER
CHERRY STREET BRIDGE, TOLEDO.

Rivets in "A"-3/4" diam. in "B" and "C"-7/8"
Countersunk rivets outside of "A" and "B"
Working Shaft "C" rests on top of "A" and "B"
Vertical seams in "A" and "B" to have inside butt straps and in "C" to have lap joints.

FIG. 9.

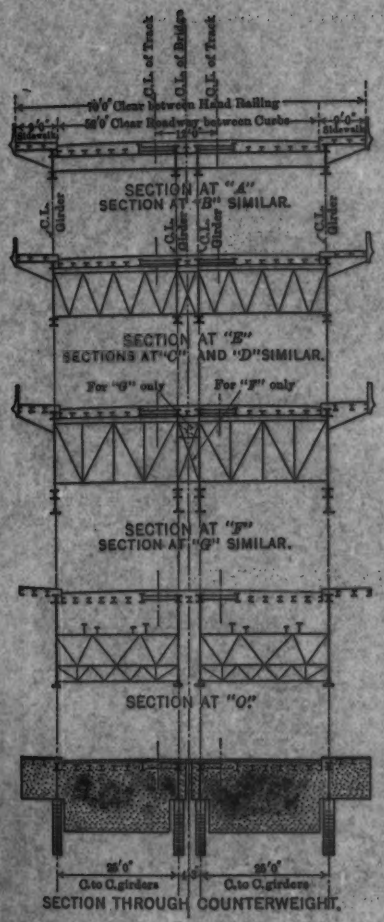
bailing had to be resorted to. After concreting to the proper elevation in the shafts, the top sections of the cylinder were lifted off and re-used.

When the coffer-dams for Piers V and VII were laid out, it became apparent that the grillages of the old piers would interfere with the construction of the new ones for their entire length, and the contractor accordingly proposed to construct the remaining piers of the bridge, except the bascule pier (Pier III), in halves, to be suitably bonded together. This was to be done without change in contract price.

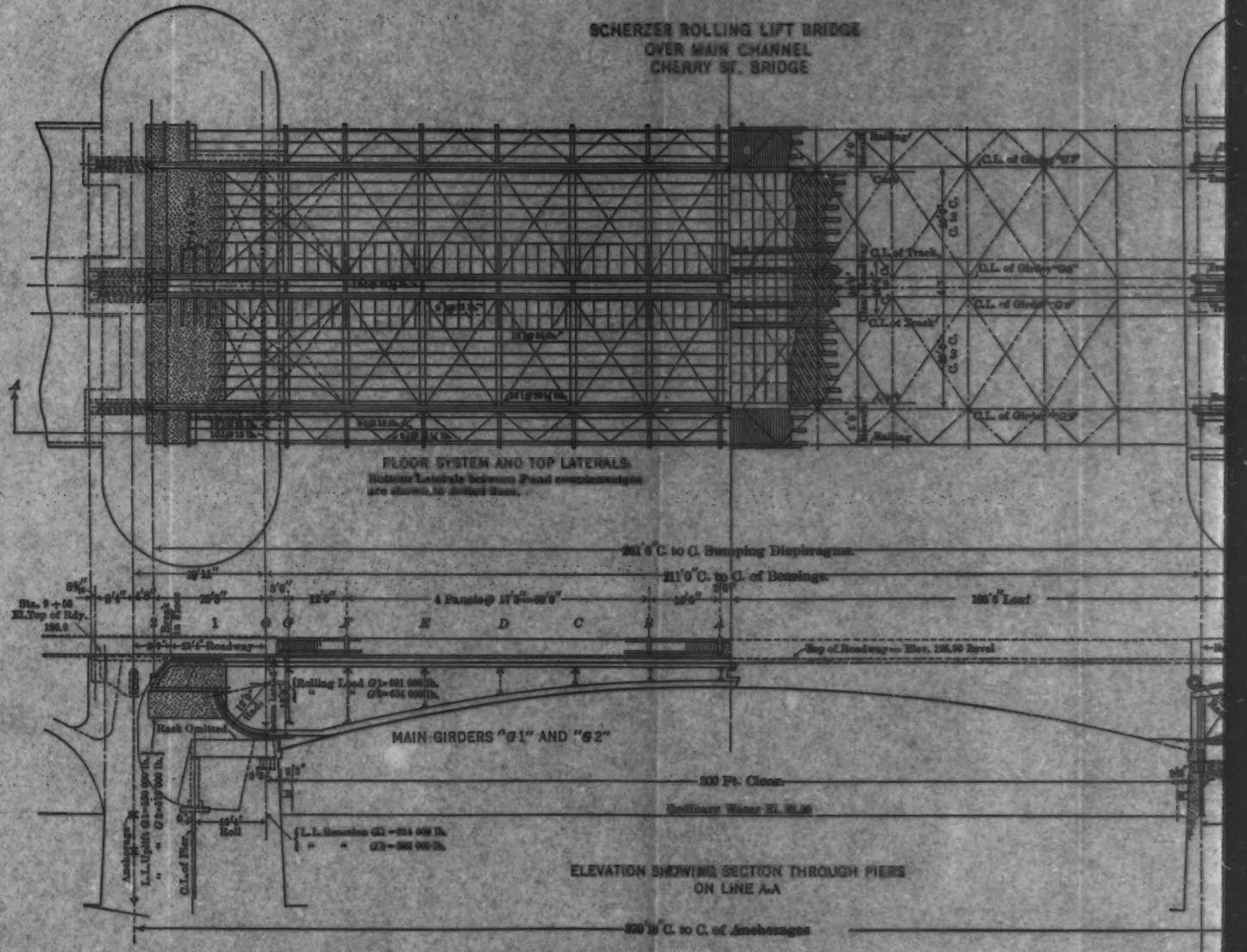
Provision was made, by grooves and projecting reinforcing rods, for bonding the second half of each pier to the first. One problem which had to be met in building the piers in halves, was that of continuing the coffer-dam after the old pier had been removed and construction of the second half of the bridge had been started. At one pier, a heavy timber bulkhead was placed from the side-wall of the dam to a groove in the side of the pier. In other cases, half a section of steel sheet-piling was concreted into the side of the pier, with provision for its removal later. In the abutment piers, a concrete bulkhead wall was built across the coffer-dam. These bulkheads permitted the pulling and re-use of most of the sheeting of the first half of the dam. The up-stream wall of the old dam was left in place to protect the bulkhead until the dam was to be extended to enclose the second half, when it was pulled, and the side-walls were continued.

Pier VII was the first in which the bottom of the coffer-dam was covered with a "blanket" of concrete before unwatering—the procedure intended at first only for the two large piers. Though Pier VIII had been successfully unwatered without this, it was thought best, as the additional cost was moderate, to place a layer of concrete under water in the remaining piers, rather than risk pumping out the dams without it, against a head of from 30 to 32 ft. For the small dams, the dimensions of which were about 25 by 48 ft., a layer 6 ft. thick was placed. This was of 1:3:5 concrete with two extra bags of cement per cubic yard of concrete in the lower part, over the tops of the cylinders, and one extra bag per cubic yard in finishing the blanket, to make up for possible loss of cement. Before starting to concrete, a diver cleaned off the tops of all the cylinder shafts, in order to insure full bearing.

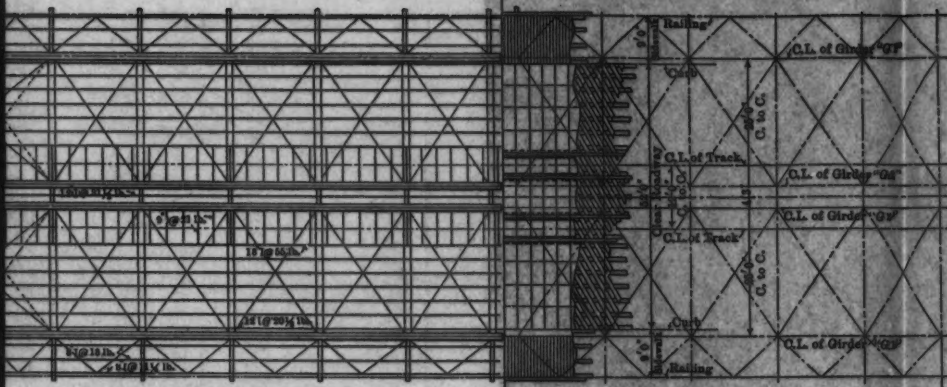
The preferred method of depositing concrete under water was by tremie, chuting the concrete directly from the spout of the floating



SCHERZER ROLLING LIFT BRIDGE
OVER MAIN CHANNEL
CHERRY ST. BRIDGE



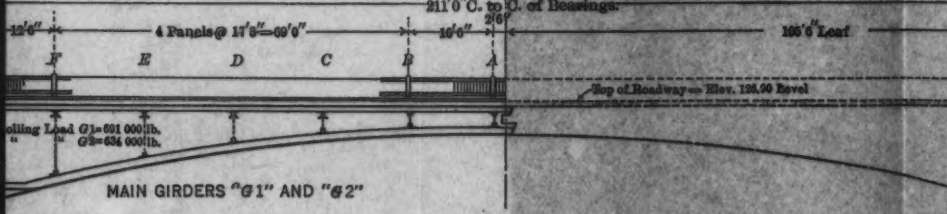
SCHERZER ROLLING LIFT BRIDGE
OVER MAIN CHANNEL
CHERRY ST. BRIDGE



FLOOR SYSTEM AND TOP LATERALS.
Bottom Laterals between F and counterweight are shown in dotted lines.

261'6" C. to C. Bumping Diaphragms.

211'0" C. to C. of Bearings.



MAIN GIRDERS "G1" AND "G2"

200 Ft. Clear.

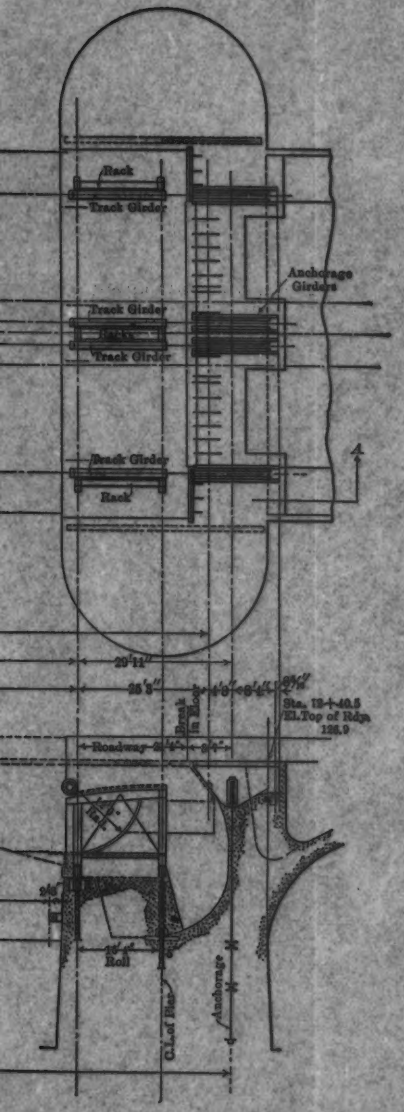
Ordinary Water El. 93.00

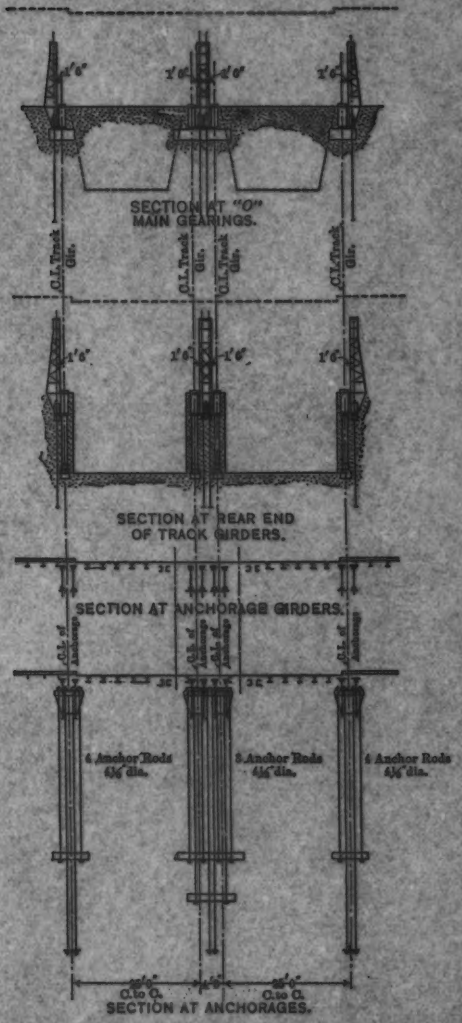
L. Reaction G1 = 314 000 lb.
" " G2 = 322 000 lb.

ELEVATION SHOWING SECTION THROUGH PIERS
ON LINE A-A

270'10" C. to C. of Anchorages

PLATE XXIX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXX, No. 1382.
CHASE ON
CHERRY STREET BRIDGE, TOLEDO, OHIO.





mixer barge into the hopper of the tremie, which was handled by either the fixed or floating derricks.

Tremie work, though practiced for years, is far from being standardized, and as it is on the details of the construction and the handling of the tube that success depends, the methods used at Cherry Street may be of interest.

The tremie pipe consisted of flanged sections of 12-in., spiral, riveted pipe, which were bolted together to make up the various lengths required. The top section was short and was riveted to a pyramidal hopper of ¼-in., sheet steel, stiffened with angles and having a capacity of about 1 cu. yd. The edges of this hopper rested on a timber frame, suspended from the derrick by slings at the four corners. The discharge pipe from the floating mixer was self-supporting, and was simply held in place over the hopper by lines, in such a way that its weight did not have to be lifted when the tube was raised. A signalman, usually accompanied by the inspector, "rode" the hopper, and gave directions to the hoisting engineman.

Three things which must be most guarded against in tremie work, under conditions such as those at Toledo, are, first, the "plugging" of the pipe; second, the loss of the charge; and third, allowing the concrete to drop through water as it leaves the pipe. The first causes serious delay, disturbance of the concrete already placed, and danger to derrick and tremie from efforts to start the concrete flowing, and, usually, in the end, loss of the charge, thus making it necessary to refill the tremie. It can be avoided by using a quite wet mix, with small aggregate where possible, and by having the pipe hoisted enough to start the charge moving just as the hopper starts to fill, instead of after it is full. There does not seem to be any advantage in having a pipe larger than that used on this work, in fact, Mr. E. B. Van De Greyn* has recently stated that an 8-in. tremie was used successfully on the foundations for the City Waterway and Puyallup River Bridges, at Tacoma, Wash.

The second trouble causes serious washing of the last of the concrete in the pipe, for, as the column of concrete nears the bottom, the water rushes up through it to fill the tube. Also, refilling the tremie always causes more or less washing out of the cement. Loss of charge can only be avoided by good team work between the signalman and

* *Engineering Record*, August 15th, 1914, p. 188.

derrickman. The lowering of the pipe to stop the flow must be started in time, so that the pipe will be completely sealed, at least when the concrete has lowered in the tube to about the water level. The pipe should be lowered gently so as not to disturb the concrete already deposited. The regulation of this speed and the proper moment for giving the signals are matters requiring considerable practice on the part of the men. Matters are greatly complicated when the tremie is handled from a floating derrick, which lurches when the load is lifted and again when it is dropped. To reduce washing in filling the tremie, a cement sack, loosely filled with straw, will act as a plunger and force the water before it as the weight of the concrete forces it through the tube. The sack will usually float to the surface after passing through the pipe.

The third trouble will cause loss of cement from the charge and churning of the deposited concrete. The pipe must be "picked", or lifted, just enough to start the flow, which will rarely bring the lower end out of the soft concrete previously deposited.

A concrete with a small aggregate is handled much more easily through a tremie than one with the larger sized stone used in open-air mass work.

In the small dams, the entire layer of 6 ft. was carried across at once, but, in Piers III and IV, two lifts of 5 ft. each were made, and the reinforcement was placed between. For that part of the blanket of Pier IV under the old span, a bottom-dump bucket was used, which was "telegraphed" (fleeted or drifted) in under from the fixed derrick to a line hung from the old bridge floor. It is a point much debated, in work with a bucket, whether a top cover protects the concrete from wash, or causes more wash by creating a partial vacuum as the charge empties. At Cherry Street, two canvas flaps, weighted on the edge, covered the surface of the concrete as the bucket was submerged. Observation of the blankets after unwatering the dams showed the expected layer of laitance and silt, from ½ in. to 2 in. thick, in the low spots of the surface, and a decided unevenness of the top surface, in spite of the care taken to get it as level as possible. Concrete under water (due to being in a denser medium) stands in slopes much steeper than would be possible with that laid in air. Samples taken from the blankets, after cleaning the surface, showed that an excellent, dense concrete had been secured. Of under-water concrete

9 000 cu. yd. were required for all the piers, of which 7 050 cu. yd. had been placed at the end of the 1912 season.

The floating mixer plant previously referred to is a type of equipment, used by several contractors along the Great Lakes, utilizing the now familiar method of tower distribution, as developed for land work. (See Figs. 10 and 11.) This means of putting concrete into place has several advantages over the floating derrick and bucket. There is less danger of damage to the forms by knocking out braces or breaking wiring, the men do not have to keep out of the way of rapidly moving buckets, and the speed is practically limited only by the capacity of the mixer. For work in pier forms, a rate of 350 batches per 9-hour shift was commonly maintained. For shorter periods, speeds up to 50 batches per hour were made, but the total for the shift was usually cut down by the minor break-downs of material-handling machinery and delays incident to the shifting of sand and stone scows. The plant consisted of boilers, a hoisting engine, a stiff-leg derrick for hoisting material to the hopper bins, a ¾-cu. yd. Koehring mixer, and the elevator tower, all mounted on a barge 32 ft. wide and 100 ft. long. The tower was 56 ft. high, with the discharge hopper mounted on a slide, so that the chute could be raised or lowered readily, without changing its slope.

For Piers I, II, and IX, as previously mentioned, piling had been substituted for the concrete cylinders. The procedure in building these piers was much the same. After excavating inside the coffer-dam to a stiff blue clay, a test pile was driven, which showed the probable penetration that would be secured. Piling was then ordered of a length to give this penetration and extend a couple of feet above the bottom. In driving, a pipe of heavy 20-in. tubing was set in the leads, with its lower end on the bottom. The piles, which were usually shorter in length than the depth of the water, were dropped into this tube and driven with a follower to the desired penetration, by a No. 1 Vulcan steam hammer. About 25% of the piles in the two abutment piers had a batter of 2 in 12 and, for these, long piles were driven with batter leads and later cut off by a diver, or cut off above the blanket after pumping.

During the work on Pier II, an accident occurred in which loss of life was narrowly averted, and gave a remarkable proof of the abuse steel sheeting will stand. Before driving the piles, the channel had

been dredged with a floating dipper-dredge to blue clay, at a depth of 30.4 ft., and then a 30 by 60-ft. coffer-dam was driven. The piling was of the fabricated channel and Z-bar type used by the contractor throughout the work, and was driven to an average penetration of 10 ft. in the blue clay. After completing the driving of the bearing piles, the blanket of concrete, 6 ft. in thickness, was deposited under water, as in the other piers, leaving a 24.5-ft. head of clear water to be pumped against. The timbering was built up in place and sunk to position, and the final wedging was done as the racks were unwatered. The dam proved to be tight, and all the leaks were readily stopped with the mixture of oak sawdust, manure, and ashes. It was found that more discharge had been provided than was needed, and some of the pumps were cut out. At 11 P. M. on June 28th, 1912, the water was down 18 ft.; 5 min. later, just after the pump men had left the hole for their lunch, the timbering suddenly gave way. Though unwitnessed, it seems most probable that the accident was caused by the jar and weight of the pumps forcing the transverse timbers from their bearings on the walings.

At first sight, the wreck (Figs. 12 and 13) seemed to be complete, but as this pier was now one of the critical points in the completion of the bridge at the promised date, it was decided to make an attempt to straighten the dam, re-timber, and pump. This hardly seemed feasible at first, as the interlock was expected to be sprung so much by the bending of the sheeting over the edge of the footing-block that the leakage would be impossible to control. Examination of the dam above water, and by a diver below, showed that, following the collapse of the bracing, the sheeting had been sharply bent at an angle of 40° over the concrete. From July 1st to July 12th, the gangs were engaged in pulling the walls back to line at the water's surface and sinking a second set of timbering, after removing the wreckage. On July 13th, pumping was started, and, to the surprise of all concerned, the water was lowered so fast that some of the pumps had to be shut down in order to allow the carpenters time to wedge up the racks as they were unwatered. By July 15th, the water was down, and the leakage was easily controlled by a single 6-in. Cameron pump.

Examination showed that the sheeting had withstood its severe punishment without damaging the interlock at any point, and was holding tight, though bent in what would have been thought to be

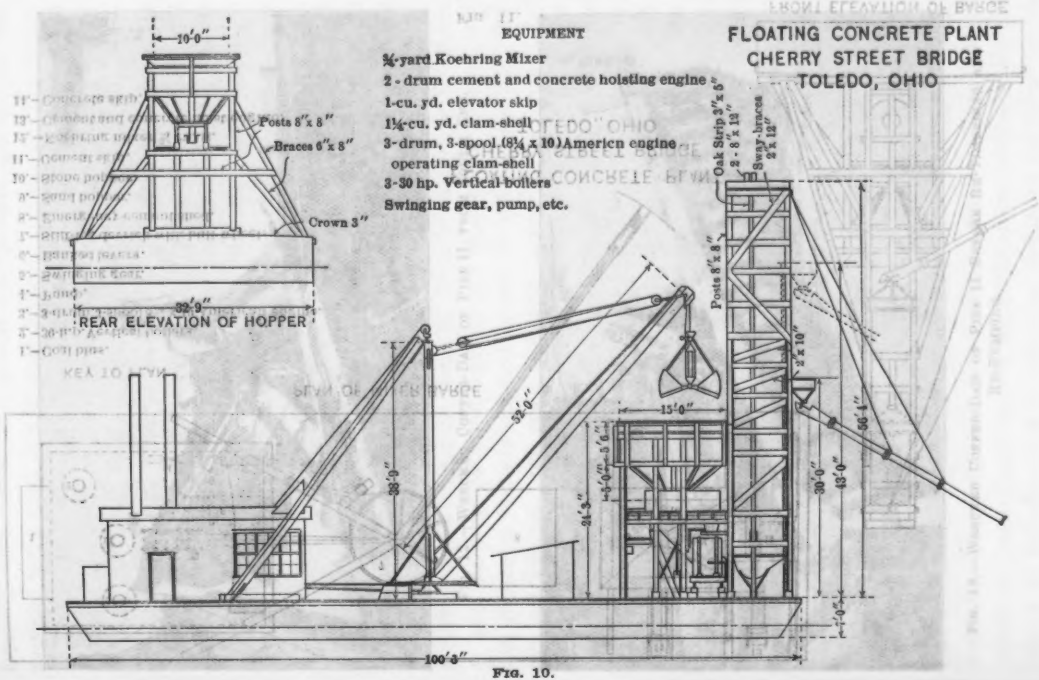
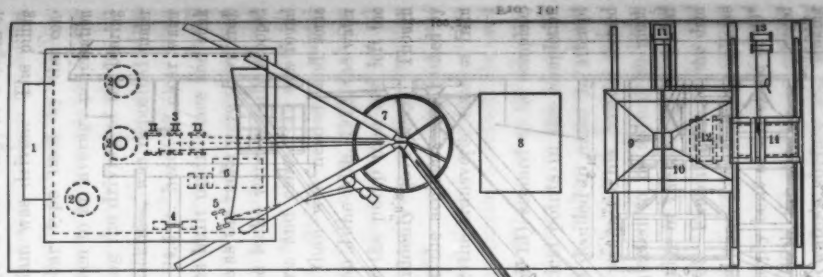

EQUIPMENT
¾-yard Koehring Mixer
2-drum cement and concrete hoisting engine
1-cu. yd. elevator skip
1¼-cu. yd. clam-shell
3-drum, 3-spool (8¼ x 10) American engine operating clam-shell
3-30 hp. Vertical boilers
Swinging gear, pump, etc.
FLOATING CONCRETE PLANT
CHERRY STREET BRIDGE
TOLEDO, OHIO
10'-0"
Posts 8"x 8"
Braces 6"x 8"
Crown 3"
32'-9"
REAR ELEVATION OF HOPPER
Oak Strip 3"x 8"
2 - 8"x 13"
Sway-braces 2"x 12"
Posts 8"x 8"
2"x 10"
32'-0"
15'-0"
38'-0"
21'-3"
30'-0"
43'-0"
36'-1"
100'-3"

FIG. 10.

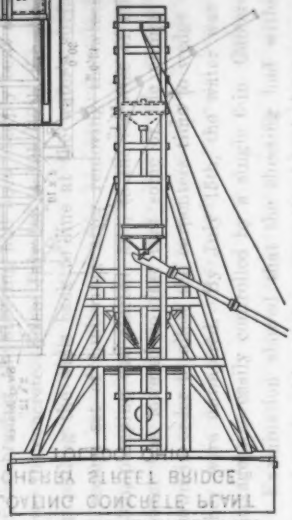


PLAN OF MIXER BARGE

KEY TO PLAN

1.—Coal bins.
2.—30-h.p. Vertical boilers.
3.—3-drum, 3-spool 8½ x 10 American engine.
4.—Pump.
5.—Swinging gear.
6.—Banked levers.
7.—Stiff-leg derrick with bull-wheel.
8.—Emergency cement shed.
9.—Sand hopper.
10.—Stone hopper.
11.—Cement skip.
12.—Koehring mixer ¾ cu. ft.
13.—Cement and concrete hoist engine.
14.—Concrete skip.

FLOATING CONCRETE PLANT
CHERRY STREET BRIDGE
TOLEDO, OHIO



FRONT ELEVATION OF BARGE

FIG. 11.

FIG. 12.—WRECKED COFFER-DAM OF PIER II, FROM SOUTH END.

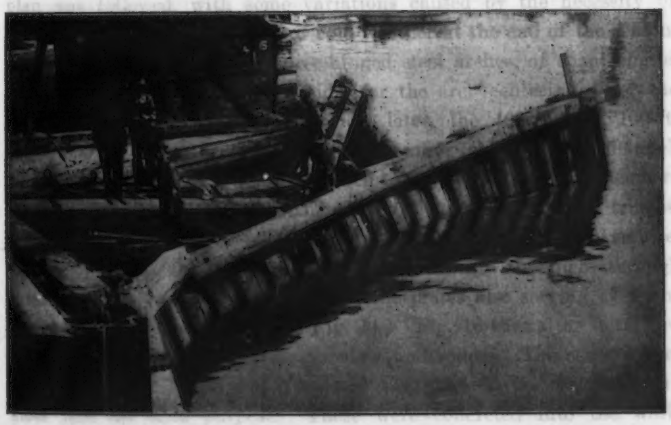


FIG. 13.—WRECKED COFFER-DAM OF PIER II BEFORE RE-TIMBERING AND RE-PUMPING.

FIG. 12.—WRECKED COFFER-DAM OF PIER II, FROM SOUTH END.

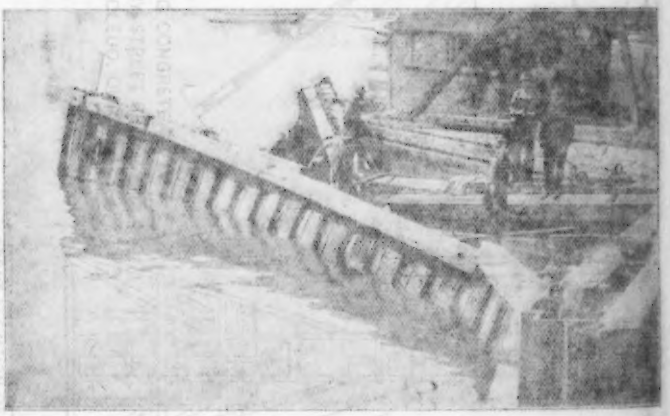


FIG. 13.—WRECKED COFFER-DAM OF PIER II BEFORE RE-TIMBERING AND RE-PUMPING.

impossibly sharp curves. Forms were rapidly built, and the pier was completed to the water line by July 19th.

Throughout this job, the contractor used about 40 000 ft. of steel sheet-piling of this type, in nine coffer-dams of an aggregate area of 21 300 sq. ft. It was easily driven and pulled, though there was considerable difficulty at times in starting to pull the piling where the footing-block had been cast under water directly against the riveted sheeting. All of it was used at least twice, and much of it oftener than that, on this first half of the job. The dams were unwatered in all cases with very little difficulty, against heads of clear water of from 20 to 25 ft.

The first work above water was started in May, 1912, when the piers on the east side of the channel had all been raised to Elevation 92, and the placing of the arch centers was begun. The contractor's scheme for the construction of the arch ribs was to cast them in successive parallel strips of about 12 ft. each, supporting the forms on steel centering until the concrete had attained its strength, and then shifting the centers sideways to cast the adjacent section. This plan was followed, with some variations caused by the necessity of rushing work on the approach of cold weather at the end of the season.

The contractor used five three-hinged steel arches, of span lengths to fit the openings on the east side, for the arch centering. For the two 108-ft. arches on the west side, later, the 108-ft. and 100-ft. centers were brought across from the other side of the channel, and the shorter one was lengthened out by bolting on members at the crown. Each center consisted of two three-hinged arch trusses, with a temporary tie to take the thrust during erection. In service, the thrust from the concrete load is carried into the piers through large bent plates at the lower hinges, the plates also serving as forms for that part of the pier. (See Fig. 14.) Fig. 16 shows the details of the centering, with the lowering wedges and rollers. The centers were supported on I-beams which crossed the piers at the water line in slots left for that purpose. These were concreted into the work, and were to be cut off later back of the face of the pier. These I-beams carried the longitudinal beams on which the rollers traveled in shifting the centers laterally. Fig. 17 shows the details of the forms for the arch ring. To secure the proper curve of the arch intrados, the outlines of semi-arches were first laid out on a large wooden platform in the

yard, and then a section of arch center was placed on its side in the correct relative position and templates made, from which the 6 by 12-in. joists were cut to the proper curve by a band-saw in the shop.

When planning the programme for pouring the arch rings, it was seen that the use of the arched centers introduced a problem not common to the ordinary timber falsework. This was the unbalanced thrust which would come on any pier from a partly or entirely completed arch on one side only. It was decided to start with the smallest arch at the east end and carry the concrete out only one-third of the distance to the crown at each pier in succession from there to the channel. This was done, the concrete fleet being shifted from side to side of the arch, so as to place fresh concrete on each surface not later than 2 hours after leaving it. As three surfaces had to be kept fresh, this moving took time and caused some risk of having a surface set before getting back to it. This, in fact, occurred at one of the points, and the rapidly hardening concrete had to be removed to form a surface normal to the thrust before the concreting could be continued. After pouring the lower part of the arches in this way, the crowns were finished as soon as the first concrete had set enough to permit the removal of the bulkheads. The floating mixer and tower handled this work very satisfactorily, as the batches could be chuted to down-spouts placed where needed through the top lagging of the forms.

After completing the first 12-ft. section, it was realized that if the plan of casting the arch ring in 12-ft. sections was adhered to, the work on the first half of the bridge could not be completed that season. It was then decided that it would be possible to cast the remaining width of 23 ft. of the half-arch barrel at one time, by increasing the load on the centers somewhat and dividing the remainder of the load between the Melan rib reinforcement and the section of arch previously completed. How this was accomplished is shown in Figs. 15 and 17. The transverse beams carried by the steel centers were prolonged with 12 by 12-in. timbers, notched to fit over the beams at one end and supported at the other end by bolts from the old concrete and in the center by stirrups which threw part of the load on the latticed ribs. As it would take more time to concrete this increased width of arch ring, the order of pouring was changed, with the idea of reducing the number of surfaces to be kept fresh. In order to do this, each

FIG. 14.—THREE-HINGED ARCH CENTERS FOR SUPPORT OF 108-FT. ARCH FORMS.

FIG. 15.—ARCH CENTERS IN PLACE READY TO COMPLETE HALF WIDTH OF ARCH IN ONE OPERATION, USING AUXILIARY NEEDLE-BEAMS.

FIG. 14.—THREE-HINGED ARCH CENTERS FOR SUPPORT OF 100-FT. ARCH FORMS.

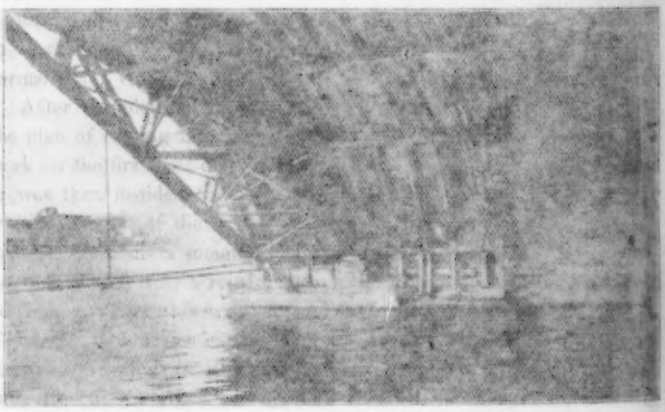


FIG. 15.—ARCH CENTERS IN PLACE READY TO COMPLETE HALF WIDTH OF ARCH IN ONE OPERATION, USING ADJUSTABLE HINGED-DECKS.

C.L. Bridge
C.L. Arch rib
2-10"In @ 23 lb.
12'0"
10'0"
B
A
C.L. Arch rib
Bottom Lateral Struts
Middle Third of Arch concreted second
Section concreted last
Bottom Lateral System
Top Lateral System
10'0" c. to c.
2 Rails @ 60 lb.
5.35" Bath Girders @ 70 lb.
12"x.48"Pl.
1st Position
2nd Position
Movements, as Planned
3rd Position
Struts of 2 rollers linked together for each shoe
Section concreted last
Lowering Device
Thread Nut
Thread right
Lock Plate
STEEL CENTERS FOR 108-FOOT ARCH
CHERRY STREET BRIDGE,
TOLEDO, OHIO.

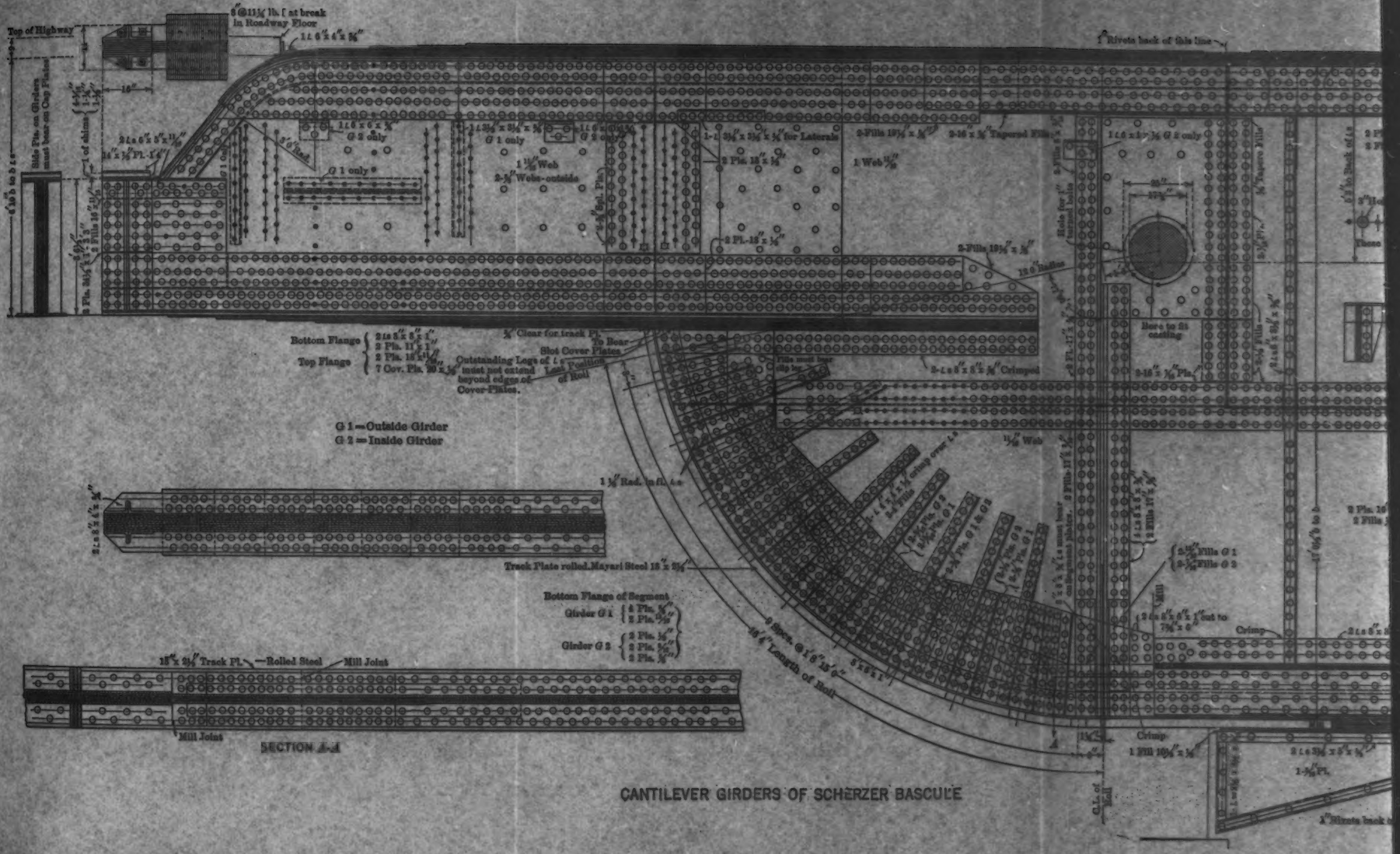
FIG. 16.

pier was poured separately up to the bulkheads at the springing line. This also embedded the ends of the latticed ribs and made them ready to act as arches in carrying their proportion of the load, later. As soon as the piers had set enough to resist the thrust of the arch centers, the haunches of the arches were poured to a point 15 or 20 ft. from the crown, the top lagging being in place before concreting was started. At this point, where the slope was about 2 in 12, the top lagging was stopped. While pouring the haunches, the crowns of the arches were loaded with concrete blocks, and, later, when pouring the crowns, the adjacent uncompleted arches were similarly loaded.

When concreting, one man worked between each pair of latticed ribs, puddling with his feet, or using a long 2 by 4-in. pole to reach the far edge of the concrete against the top lagging. Particular care was taken in this way to avoid the formation of rock pockets. The concrete came into the forms through hoppers placed at regular intervals along the top lagging. The plant was shifted from side to side of the arch at intervals of about 2 hours, so that fresh concrete was placed on the old before it had set more than 2½ hours. At this age, in the weather during which most of the work was done, the surface of the concrete was still soft enough to insure a good bond.

One of the chief difficulties encountered in the building of the spandrel walls, after the arch rings were complete, was the placing of the facing concrete which was used in order to secure an attractive texture of the concrete surface after bush-hammering. This mixture was of blue-gray and white limestone chips with a mortar of stone screenings and sand. Facing "irons" were first attempted, but were not feasible on account of the wet mixtures in use and a tendency to leave voids or rock pockets in the face of the work. The final method adopted was to use a liberal quantity of the mixture and pour it from buckets against the forms, keeping its level always slightly higher than the surface of the concrete. The rock pockets on the face that resulted from the attempt to use facing irons were filled later by use of the cement gun.

The arches were not ready for water-proofing until very late in the season, for the best results. The five-ply water-proofing over the arch barrels and inside of the spandrel wall, however, was placed as rapidly as possible after the completion of the concrete, so that the back-filling might be commenced. The order of work was to clean


Top of Highway
Bottom Flange
Top Flange
G 1 = Outside Girder
G 2 = Inside Girder
Track Plate rolled Mayari Steel 18" x 2¼"
Bottom Flange of Segment
Girder G 1
Girder G 2
18" x 2¼" Track Pl.
Rolled Steel
Mill Joint
SECTION A-A
Length of Roll
CANTILEVER GIRDERS OF SCHERZER BASCULE

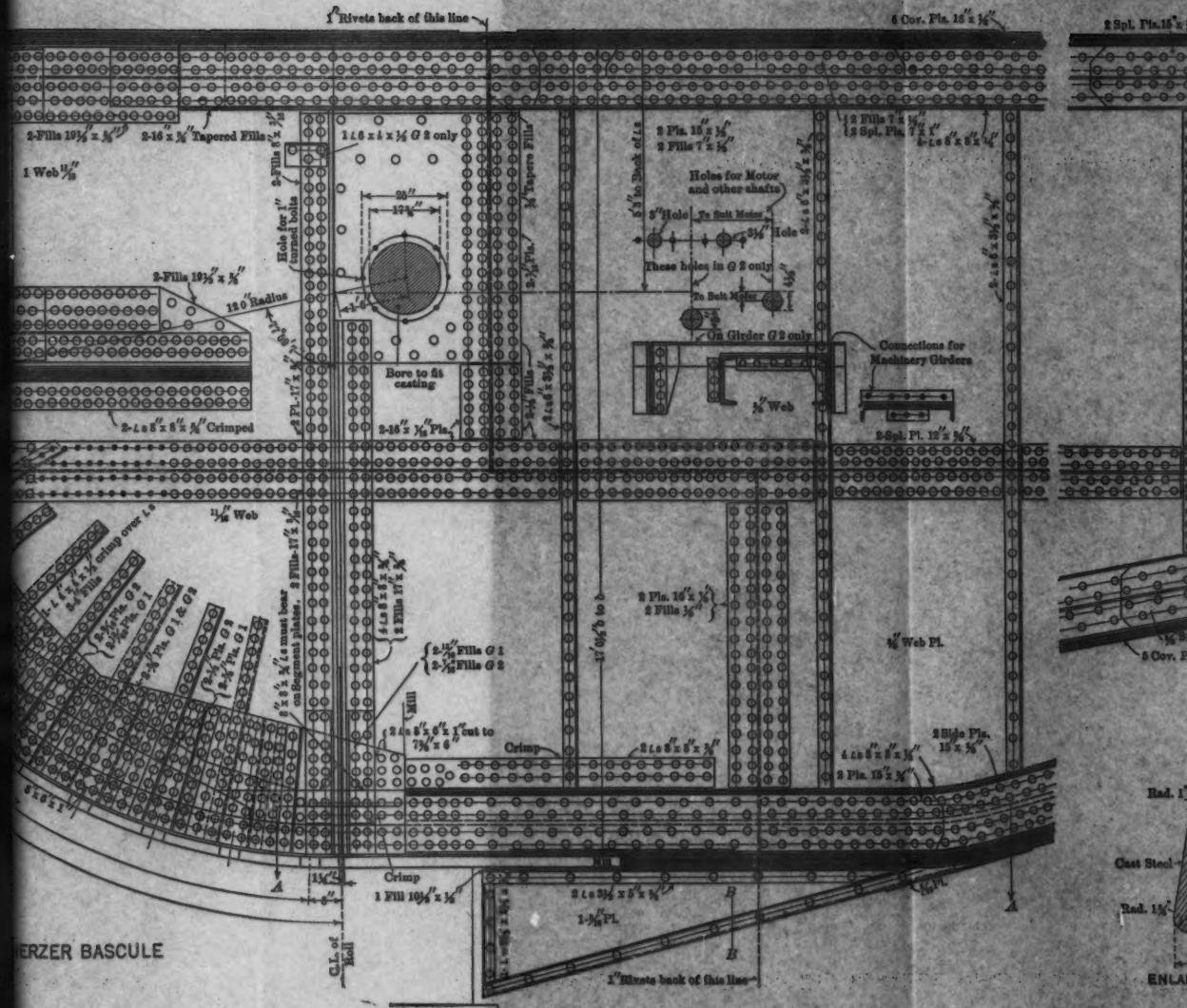

1" Rivets back of this line
6 Cov. Pls. 15" x ½"
2 Spl. Pls. 15" x
2-Fills 19½" x ⅜"
2-16 x ⅜" Tapered Fills
1 Web 11/16"
2-Fills 8" x ⅜"
Tapered Fills
1 L 6 x 4 x ½ G 2 only
2 Pls. 15" x ½"
2 Fills 7" x ½"
2 Fills 7" x ½"
2 Spl. Pls. 7" x 1"
2-L s 8" x 8" x 1"
Holes for Motor and other shafts
5 3/16" to Back of L s
2" Hole
To Suit Motor
2½" Hole
These holes in G 2 only
To Suit Motor
On Girder G 2 only
Connections for Machinery Girders
2-Fills 19½" x ⅜"
120° Radius
Hole for 1" turned bolts
28"
17¾"
Bore to fit casting
2-L s 8" x 8" x ⅝" Crimped
Pl. 17" x ⅜"
2-16" x 7/16" Pls.
2-Fills 8" x ⅜"
2-L s 6" x 3½" x ⅜"
⅜" Web
2-Spl. Pl. 12" x ⅜"
11/16" Web
2-L s 4" x 3½" x ⅜" crimp over L s
2-Fills 17" x ⅜"
4-L s 8" x 8" x ⅜"
2-Fills 17" x ⅜"
2 Pls. 16" x ⅜"
2 Fills ⅜"
17' 0⅛" to b
⅜" Web Pl.
2-½" Fills G 1
2-⅜" Fills G 2
Mill
2 L s 8" x 6" x ⅜" cut to 7¾" x 6"
Crimp
2 L s 8" x 8" x ⅜"
4 L s 8" x 8" x ½"
2 Pls. 15" x ⅜"
2 Slide Pls. 15" x ⅜"
Rad. 1'
Cast Steel
Rad. 1½"
1½"
6"
Crimp
1 Fill 10⅛" x ½"
2 L s 3½" x 3½" x ½"
1-7/16" Pl.
B
B
C.L. of Roll
1" Rivets back of this line
ERZER BASCULE
ENLA

PLATE XXX.
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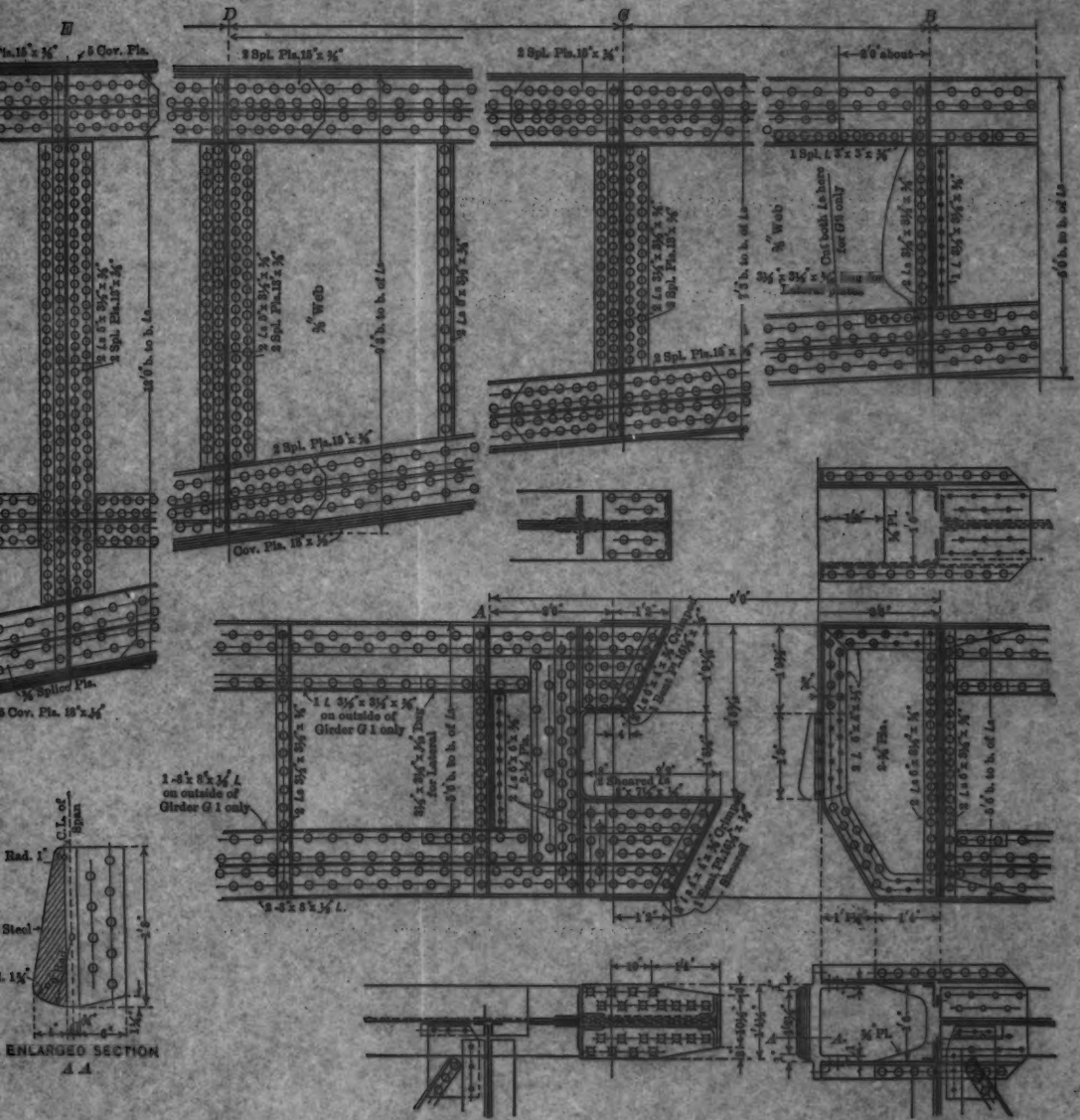
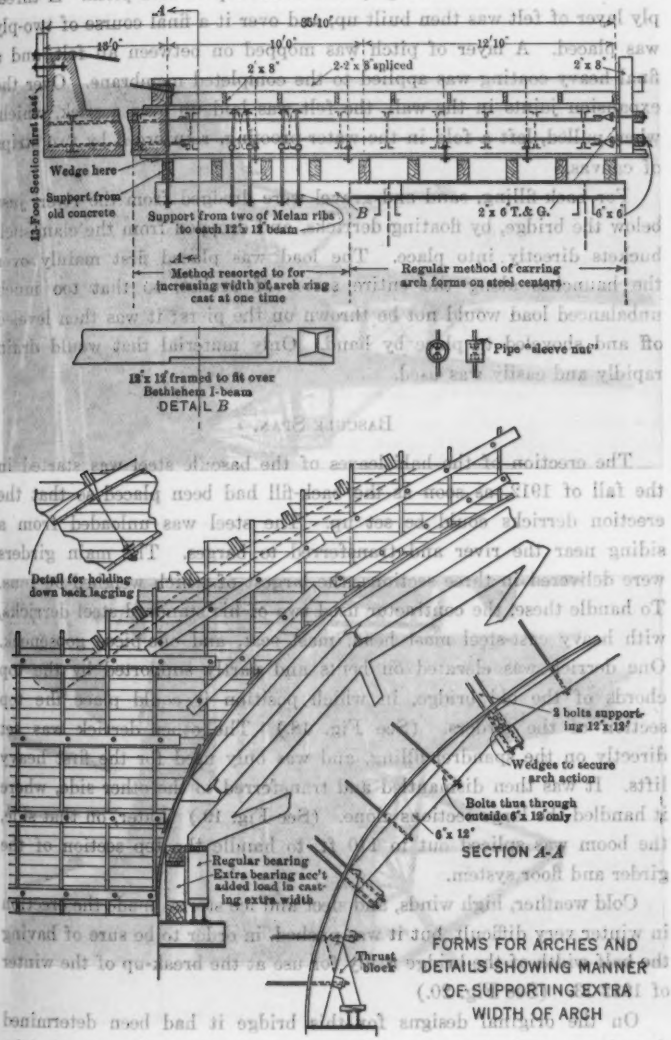
CHASE ON
CHERRY STREET BRIDGE, TOLEDO, OHIO.





FORMS FOR ARCHES AND DETAILS SHOWING MANNER OF SUPPORTING EXTRA WIDTH OF ARCH

Fig. 17.

the concrete surface thoroughly and then mop it with pitch. A three-ply layer of felt was then built up, and over it a final course of two-ply was placed. A layer of pitch was mopped on between all felt, and a final heavy coating was applied to the completed membrane. Over the expansion joints in the wall, the felt was laid up over a stick, which, when pulled, left a fold in the water-proofing, reinforced by two strips of canvas.

For back-filling, sand and gravel were dredged from the river, just below the bridge, by floating derricks, and dropped from the clam-shell buckets directly into place. The load was placed first mainly over the haunches along the entire series of arches, so that too much unbalanced load would not be thrown on the piers; it was then leveled off and shoveled to place by hand. Only material that would drain rapidly and easily was used.

Bascule Span.

The erection of the half leaves of the bascule steel was started in the fall of 1912, as soon as the back-fill had been placed so that the erection derricks could be set up. The steel was unloaded from a siding near the river and transferred to barges. The main girders were delivered in three sections, the largest of which weighed 36 tons. To handle these, the contractor used two of his standard steel derricks, with heavy cast-steel mast head, mast seat, and one-piece gooseneck. One derrick was elevated on bents and partly supported by the top chords of the old bridge, in which position it could place the top section of the girders. (See Fig. 18.) The other derrick was set directly on the spandrel filling, and was only used for the first heavy lifts. It was then dismantled and transferred to the other side, where it handled the large sections alone. (See Fig. 19.) Later, on that side, the boom was spliced out to 110 ft. to handle the top section of the girder and floor system.

Cold weather, high winds, and sleet and ice storms made the erection in winter very difficult, but it was pushed, in order to be sure of having the half width of the bridge ready for use at the break-up of the winter of 1912-13. (See Fig. 20.)

On the original designs for this bridge it had been determined that the movable span was to be a double-leaf bascule, supported by four lines of cantilever girders the lower flanges of which curved to

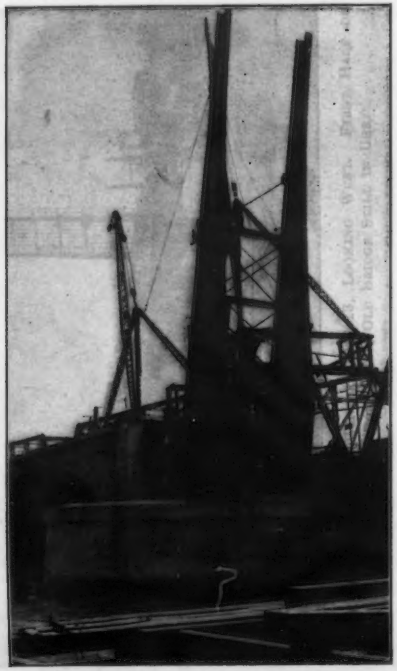


FIG. 18.—ERECTION OF FIRST HALF OF SCHERZER BASCULE IN OPEN POSITION, PIER IV.

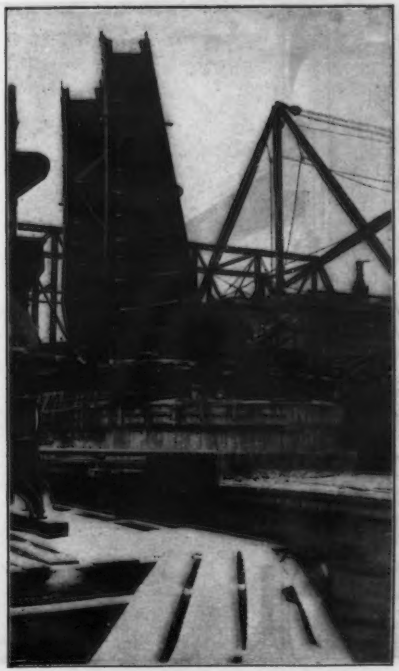


FIG. 19.—ERECTION OF BASCULE GIRDERS, PIER III.

SEE STREET OF OTHER LOCATION, LESS IN
LOT 13. ERECTION OF RAIL LIFTS ON GIRD-

LESS IN
ERECTION OF LIFTING GIRDERS

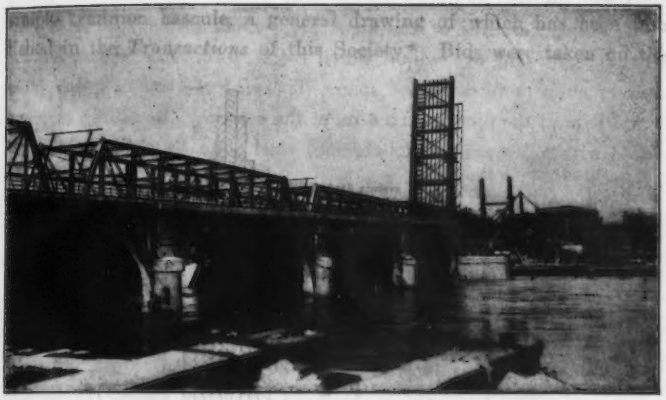


FIG. 20.—PROGRESS VIEW, WINTER, 1912-13, LOOKING WEST. FIRST HALF OF BRIDGE NEARLY COMPLETED, OLD BRIDGE STILL IN USE.

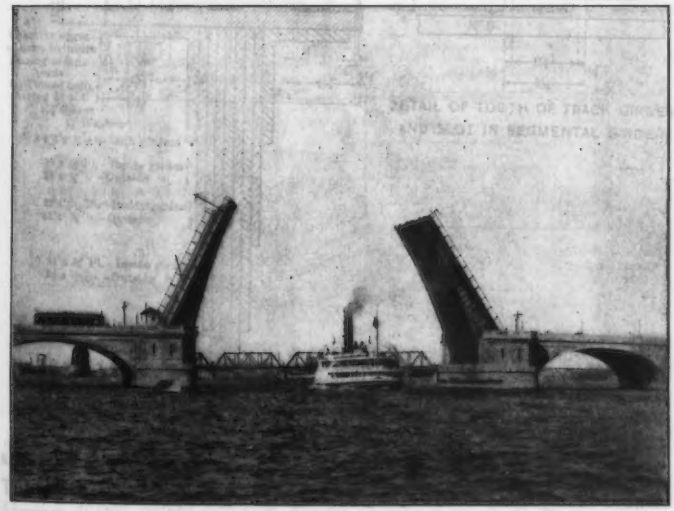
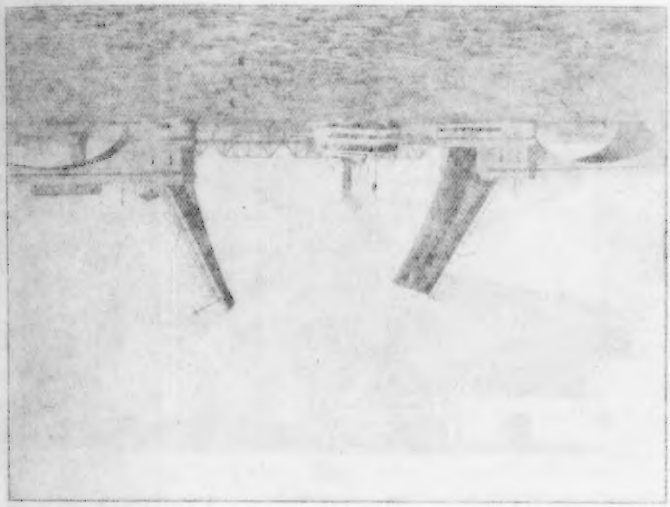


FIG. 21.—CHERRY STREET BASCULE SPAN IN OPERATION. PASSING A LAKE BOAT.



an arch. The clear opening is 200 ft. The official design was for a simple trunnion bascule, a general drawing of which has been published in the *Transactions* of this Society.* Bids were taken on this

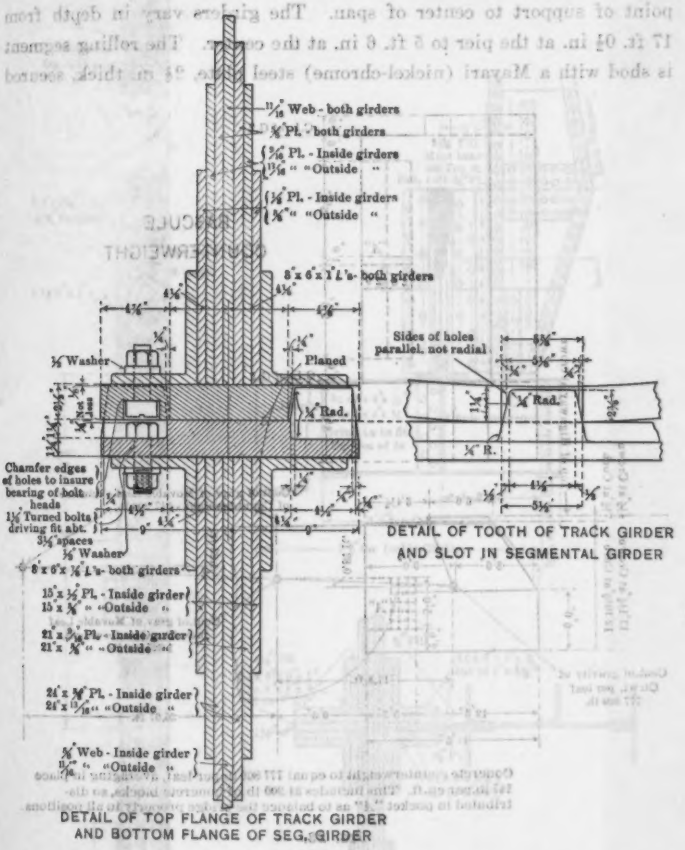
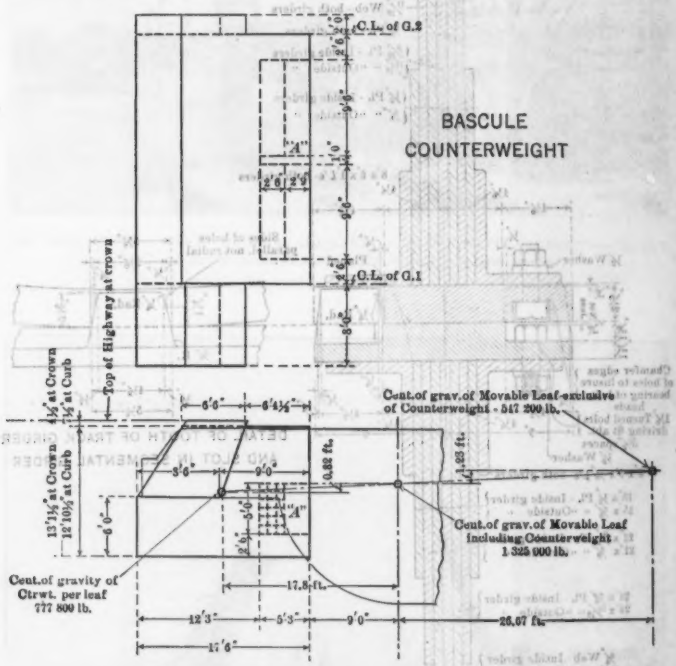


FIG. 22.

and alternate designs submitted by various movable bridge patentees. The Scherzer Rolling Lift Bridge Company was the successful bidder.

* Discussion by Wilbur J. Watson, M. Am. Soc. C. E., of "Movable Bridges", by C. C. Schneider, Past-President, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LX (June, 1908), Plate XXXII.

and that Company designed the present bridge. Plate XXIX shows the principal features of the structure. Each girder (Plate XXX) is 131 ft. 6 in. long over all, of which 105 ft. 6 in. is the distance from point of support to center of span. The girders vary in depth from 17 ft. 0½ in. at the pier to 5 ft. 6 in. at the center. The rolling segment is shod with a Mayari (nickel-chrome) steel plate, 2½ in. thick, secured



Concrete counterweight to equal 777 800 lb. per leaf, averaging in place 147 lb. per cu. ft. This includes 34 900 lb. of concrete blocks, so distributed in pocket "A" as to balance the bridge properly in all positions.

FIG. 23.

to the flanges by turned bolts. (See Fig. 22.) The track plates (Plate XXXI) are of cast steel. In the half leaf first erected the two girders are 25 ft. from center to center, and the inside one is 2 ft. 1½ in. from the center line of the bridge, making possible the erection of the second half of the bridge while the first is in use. To counter-

PLATE XXXI.
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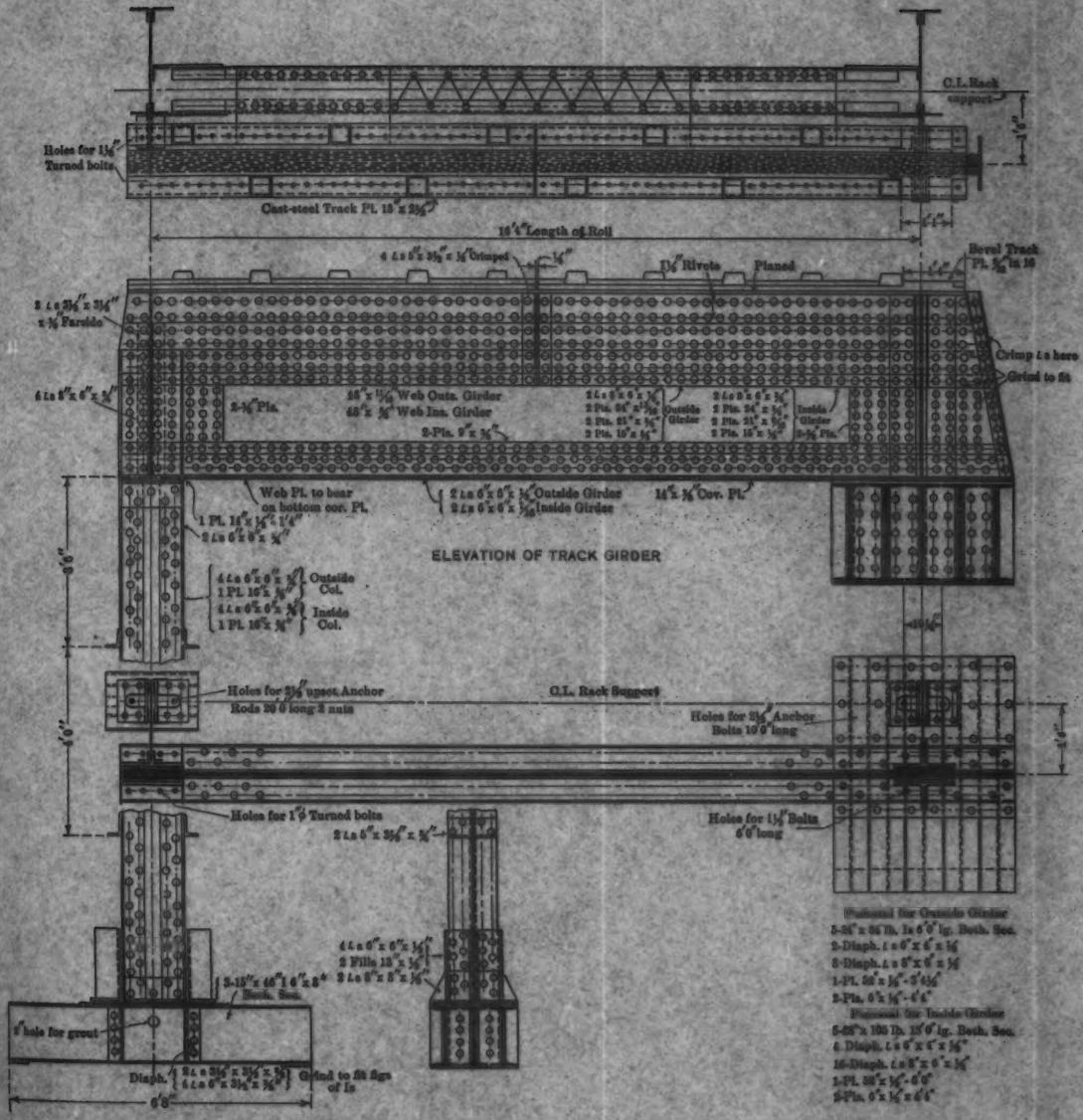
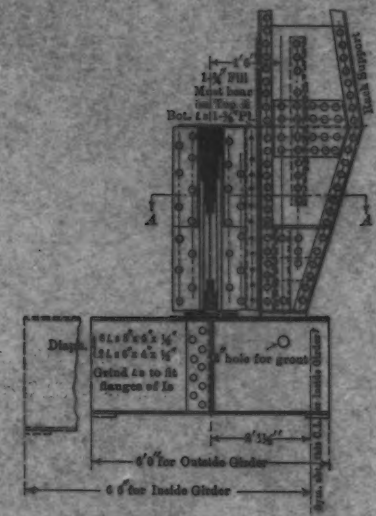
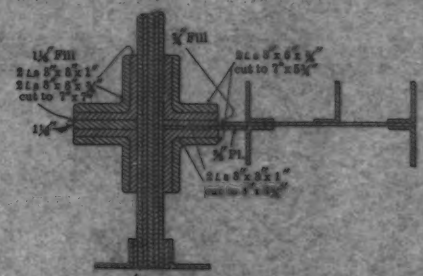
DETAILS OF TRACK GIRDER



Pedestal for Outside Girder
3-24" x 36 lb. Is 6'0" lg. Both. Sec.
2-Diaph. ∠s 6" x 4" x ⅜
8-Diaph. Ls 6" x 6" x ⅜
1-Pl. 26" x ⅜" - 5'6½"
2-Pls. 6" x ⅜" - 4'4"

Pedestal for Inside Girder
3-28" x 100 lb. 13'0" lg. Both. Sec.
4 Diaph. ∠s 6" x 4" x ⅜"
16-Diaph. Ls 8" x 6" x ⅜"
1-Pl. 32" x ⅜" - 6'0"
2-Pls. 6" x ⅜" x 4'4"





SECTION A-A (ENLARGED)

balance the dead load of each girder, 387 tons of concrete counterweight is provided, as shown by Plate XXIX and Fig. 23. The live-load uplift at the rear end of each girder is transferred to four round steel rods extending to an anchorage in the masonry of the pier. Details of the bascule floor are shown on Plate XXXII.

A 75-h.p., railway-type motor is mounted between each pair of girders to operate the structure. From this motor, a train of gears reduces the speed from 560 to 1.1 rev. per min. at the drive pinion. Each leaf can be operated from its own side of the channel, or both can be worked together by the operator in the main control house, the successive positions of both leaves being indicated there by signals.

Completion of the Work.

This account carries the history of the bridge up to the spring of 1913, at which time the writer's connection with the work ceased. The construction work in the season of 1913 was practically in all details a duplication of that done previously, after the removal of the old steel spans permitted building the second half of the structure. With the old bridge out of the way, and profiting by the experience of the preceding years of work and with an organization trained to the methods in use, far better time was made in this working season. Except that both halves of Piers III, IV, and VI, were complete at the beginning of the season, practically the same amount of work was done between March and December 31st, 1913, as in the preceding 2½ years. To avoid the loss of time incident to casting the arch rings in parallel sections, an adaptation of the method followed during the preceding year, to increase the width of strip cast, was used, together with some pile falsework, to permit of casting the entire remaining width of the arch ring at one time. Erection of the bascule was started before the end of the year, and by January 1st, 1914, the work of the contractor for the concrete construction was practically done and most of the bascule steel was in place. The fact that the channel was closed by ice allowed the erection sub-contractor to place the girders for the second half of the span in their horizontal position and connect them to the first half leaf while it remained closed.

Quantities and Cost.

The unit prices bid by the C. H. Fath and Son Construction Company, on March 3d, 1910, for this work were as follows:

Removing old masonry from piers and old rip-rap from ice-breakers and around old piers, per cubic yard	$1.50
Removing timber grillage from under old piers, per 1 000 ft. b. m.	15.00
Pulling old piles, each	3.00
Excavation and removal of sand, earth, and gravel inside coffer-dams and above cylinder piers, per cubic yard	1.50
Clay in coffer-dams or in cylinders, per cubic yard	1.65
Hardpan in cylinders, per cubic yard	1.80
Rock in cylinders, per cubic yard	3.50
Steel rings in cylinder shafts (not removed*), per pound	0.07
Steel rings in cylinder shafts (removed*), per pound	0.04
Timber lagging for cylinder shafts, in place, per 1 000 ft. b. m.	70.00
Old rails for reinforcing piers, abutments and cylinders, per long ton	30.00
Class "AA" concrete, for slabs, beams, girders, columns, etc. 1:2:4 mix	$10.80 per cu. yd.
Class "A" concrete, for arch rings. 1:2:4 mix	19.00 " " "
Class "B" concrete, for spandrel walls, etc. 1:3:6 mix	10.80 " " "

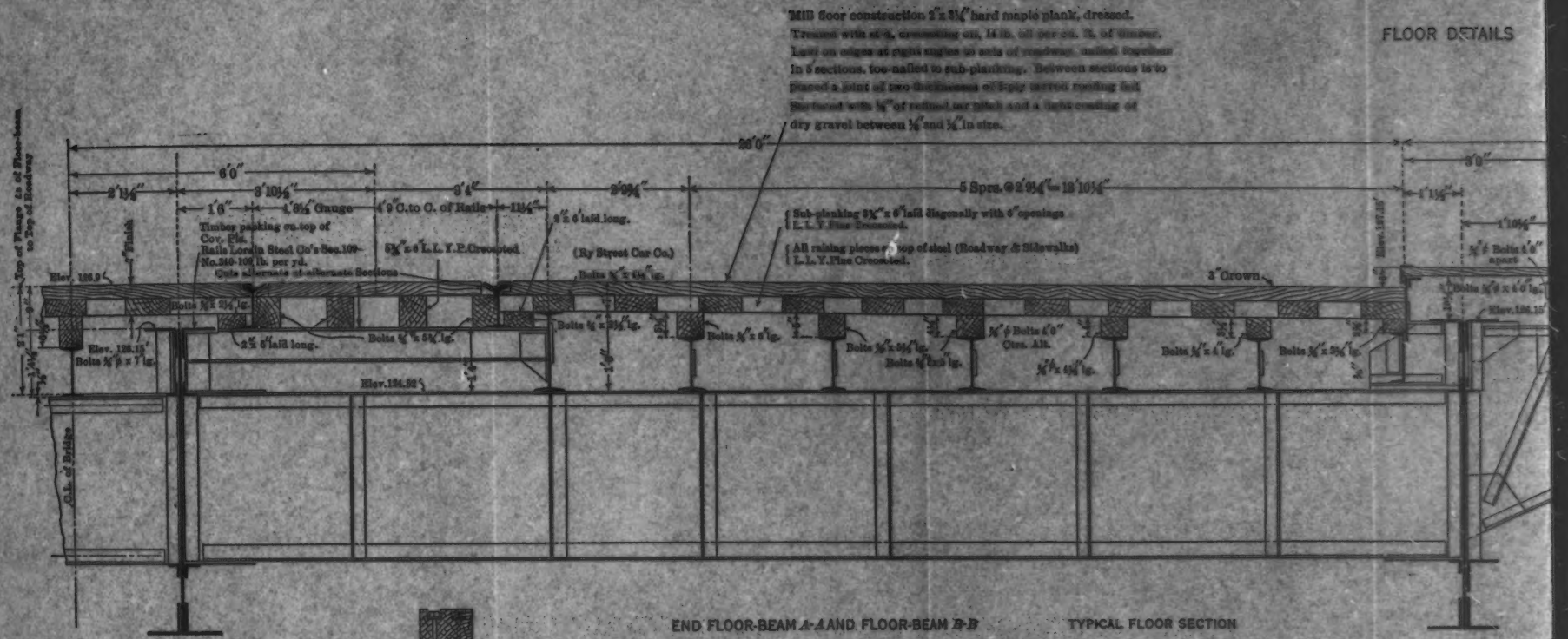
(In construction this was actually changed to a 1:2:4 mix, using small stone instead of the 2½-in. limit specified.)

Class "C" concrete for piers above mean water level. 1:3½:7 mix	$7.35 per cu. yd.
Class "D" concrete for mass work in piers below mean water level (with embedded stone permitted up to 33%) 1:3½:7 mix	6.00 " " "
For the same, in cylinders	5.25 " " "

* * *

* Provision was made for using metal lining for the cylinders, if necessary, at a price of 7 cents per lb. if left in place. When the method of foundation was changed, this price was applied to the steel cylinders, with a deduction of 4 cents per lb. for steel removed.

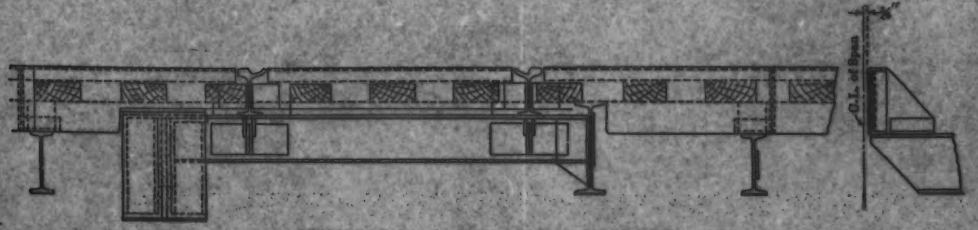
FLOOR DETAILS



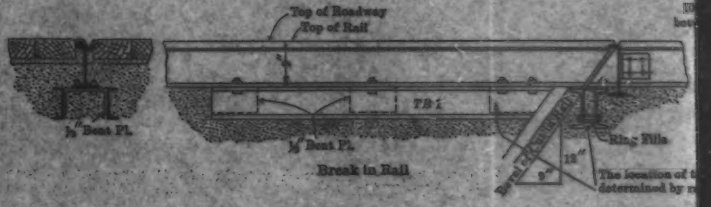
END FLOOR-BEAM *A-A* AND FLOOR-BEAM *B-B*

TYPICAL FLOOR SECTION

TYPICAL SEC. OF NAILING STRIP

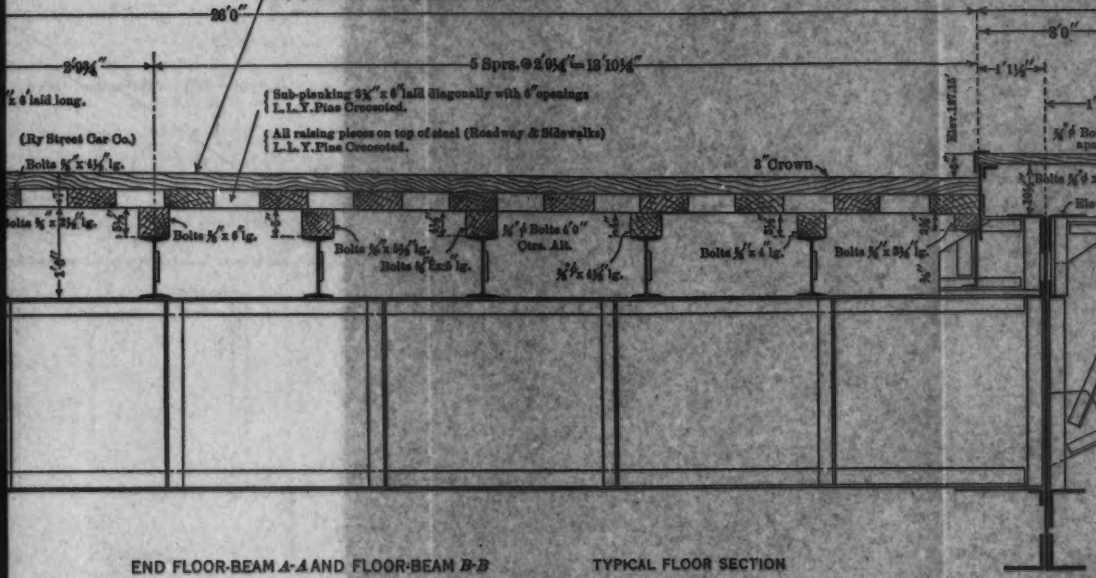


SECTION AT C.L. OF SPAN-SHOWING FINISH OF FLOOR.



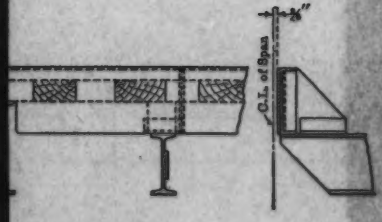
Mill floor construction 2"x 3¼" hard maple plank, dressed. Treated with st'd. creosoting oil, 14 lb. oil per cu. ft. of timber. Laid on edges at right angles to axis of roadway, nailed together in 5' sections, toe-nailed to sub-planking. Between sections is to placed a joint of two thicknesses of 3-ply tarred roofing felt Surfaced with ⅛" of refined tar pitch and a light coating of dry gravel between ⅛" and ¼" in size.

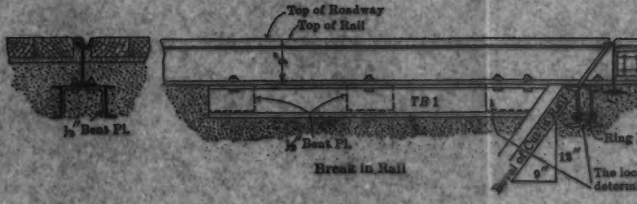
FLOOR DETAILS



END FLOOR-BEAM *A-A* AND FLOOR-BEAM *B-B*

TYPICAL FLOOR SECTION

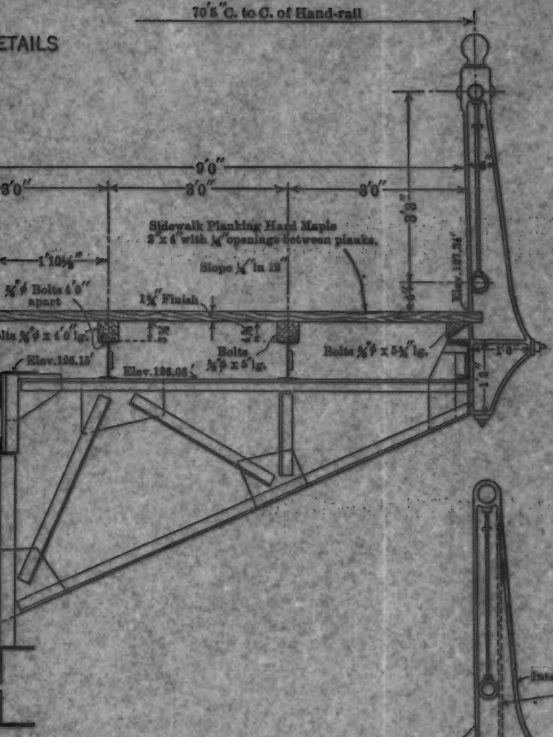


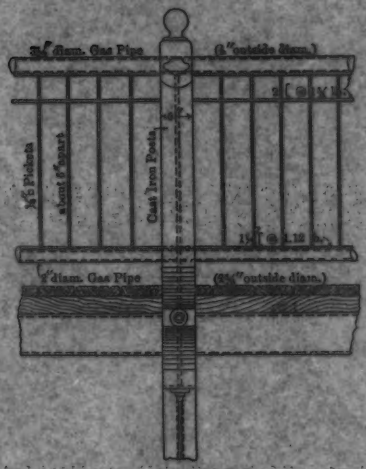


NISH OF FLOOR.

PLATE XXXII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXX, No. 1362.
CHASE ON
CHERRY STREET BRIDGE, TOLEDO, OHIO.

ETAILS

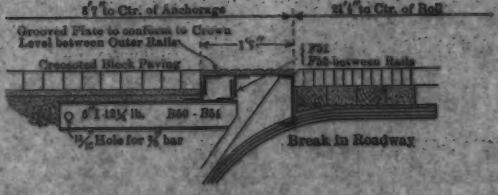




Post every Panel Point

Intermediate Post every Half Panel.
" " near Pier Points.

The location of these pieces to be determined by rails.



To January 1st, 1914, the amount expended for various items in the construction of this bridge was as follows:

Maintenance of old bridge and removal of old masonry, etc.		$42 196.80
Construction and maintenance of the nine coffer-dams, lump sum		181 500.00
Excavation, under four classifications (with 75 cu. yd. of rock)	31 625 cu. yd.	49 075.05
Timber piling, in place	35 779 lin. ft.	14 311.60
Steel and wood lagging for foundation cylinders		103 595.82
Mass concrete in cylinders, piers, etc., in three classifications	19 795 cu. yd.	119 596.05
Concrete in arch rings, 1:2:4 mix	8 916.6 cu. yd.	177 440.34
Concrete in spandrel wall, pier faces, and viaduct, in three classifications	7 519 cu. yd.	88 705.00
Concrete in foundation blankets, placed under water, including cost of extra cement	9 420.0 cu.yd.	63 024.30
Concrete parapet wall, total lump sum = $10 000		7 800.00
Finishing surface of bridge, total lump sum = $6 000		3 000.00
Steel reinforcement	732 tons	54 210.00
Water-proofing arches and spandrel walls, lump sum		20 000.00
Spandrel filling	11 800 cu. yd.	9 440.00
Setting bascule anchorage	197.5 tons	5 925.00
Miscellaneous items		3 785.00
		$943 605.16
Contract cost of bascule		132 980.00
		$1 076 585.16
All other expenses, May, 1906, to January 1st, 1914		89 082.23
Total cost to January 1st, 1914		$1 165 667.39

These items may be roughly summarized as follows:

Substructure work, new	$518 000	
Removal of old bridge masonry, etc.	32 000	
		$550 000
Concrete superstructure	$380 000	
Steel bascule span	138 000	
		518 000
Miscellaneous expense		98 000
		$1 166 000

It has been estimated by those in charge of the work since January, 1914, that the cost to complete all unfinished work after that date, exclusive of the towers (the fate of which at present hangs in the balance) would be about $15 000, besides claims not yet adjusted amounting to about $40 000.

The original design of the Cherry Street Bridge was made by the Osborn Engineering Company, of Cleveland, Ohio, under the direction of Wilbur J. Watson, M. Am. Soc. C. E., at that time the Bridge Engineer of that Company.

In July, 1910, Ralph Modjeski, M. Am. Soc. C. E., was appointed Consulting Engineer in Charge. No changes were made by him in the general features of the design, with the exception of those involved in the methods of securing the foundations, already described, certain revisions in the proportions of the arch rings and the design of the reinforced concrete viaduct, and those changes incident to the use of the Scherzer instead of the trunnion bascule. All construction work was done under his supervision. In April, 1911, Mr. Arnold W. Brunner was retained by the City as Consulting Architect, and the external treatment of the structure was handled by him, in conjunction with Mr. Modjeski.

Gustavus A. Gessner, M. Am. Soc. C. E., was Resident Engineer from the commencement of work until December, 1911, and W. R. Weidman, M. Am. Soc. C. E., from that date until January, 1914. Mr. M. B. Case and the writer were Mr. Weidman's Assistants during the year and a half preceding June, 1913. The main contract, except for the bascule span, was in the hands of the C. H. Fath and Son Con-

struction Company, until the season of 1913, when the National Foundation and Engineering Company was organized to complete the work. Mr. M. J. Comer was Superintendent. The contract for the Scherzer span was assigned to the Toledo Bridge and Crane Company, and the steel was erected by the Kettler-Elliott Erection Company.

DISCUSSION

Mr. Godfrey.

EDWARD GODFREY,* M. AM. SOC. C. E. (by letter).—This paper is another exemplification of the fact that engineers are capable of taking up and solving the big problems of construction, and for this reason it is a very valuable addition to engineering literature.

The paper also illustrates the fact that some of the smaller problems, the so-called details of design, are not given the thought and attention that their importance deserves. Engineering science is not balanced in this respect. The big things are worked out to the exclusion of details that are of equal, and sometimes of greater, importance.

It was a small thing that caused the Quebec Bridge to fall, a mere detail, the entire absence of bracing in a great traveler 217 ft. high. A great deal of work and thought were devoted to the subject of secondary stresses, which in material of the toughness of steel is of little or no consequence; but the simple matter of preventing the lateral swaying of 1 000 000 lb. of steel, held more than 200 ft. in the air, was given no consideration whatever.

The writer checked a set of details for structural work of two theaters. There were twenty-two types of errors, some of them repeated many times: Cantilevers had little or no provision for bending at the section of greatest moment; some had provision for tension but none for compression; curved girders had nothing to prevent them from flopping down, as did a large one in the Orpheum Theatre, in New York City, pulling the entire structure with it. Other errors were found. This was the work of one who held himself to be a capable structural engineer.

With practically no exception, great structural failures (excluding dams) have been due to lack of bracing or lack of attention to details of design.

Fig. 3 shows that the columns of the approach viaduct of this bridge are of square and oblong shape, having upright rods and so-called hoops 12 in. apart. A rectangular hoop is the latest thing in "squaring the circle". The writer has publicly described and shown a photograph of a column of this kind that broke at 200 lb. per sq. in. due to the mere shock of a near-by wreck. These columns, by the writer's calculation, are loaded to 400 or 500 lb. per sq. in., and belong in the class that has characterized nearly all the many reinforced concrete wrecks, where the columns are broken up into short chunks or ground to a shapeless mass.

It is true that the Report of the Joint Committee allows columns of this sort, with only wires enough to hold the upright rods in place during the hardening of the concrete; it also allows plain concrete columns. If these columns were not more than 24 ft. high, and had

* Pittsburgh, Pa.

no steel whatever, they would satisfy that Report, but the writer would hate to live under the viaduct. Mr. Godfrey.

The columns of this viaduct were not even chamfered on the corners. T. L. Condron, M. Am. Soc. C. E., says of the rodded columns of the Edison Building:*

"In places where the heat was not sufficient to destroy the insulation on electric wires strung through the ceiling, square columns were spalled almost as badly as those in the rest of the building where the heat was exceedingly great."

These facts and the failure of the column at 200 lb. per sq. in. indicate clearly that there is an intensity of strain in such columns ready to break out in failure from small contributing causes. All know what happened to the columns of the Edison Building, and how so many of them failed, and how so large a portion of one of the buildings collapsed on account of such failure, and also that such columns have been roundly condemned.

The rods in the Edison Building were 3 in. from the surface, and yet many of them broke out and split the columns from top to bottom, both those at the corners of columns and those in the middle of the sides. They did not have ties, but it is unthinkable that a few square hoops could redeem such members and make them fit to sustain a structure. The column which failed at 200 lb. per sq. in. was oblong, and had regulation ties, such as the oblong columns of this viaduct. The concrete was good and well seasoned.

The time will come when engineers must realize that not everything stated in books and committee reports is safe to follow, and the so-called destructive critic must be reckoned with. The writer has been publicly condemning columns of this kind for ten years, and he has yet to hear any better argument in favor of them than this: "Mr. Godfrey's views are not in agreement with the majority of the Profession."

Circular or octagonal columns with upright rods (not for compression but for transverse strains) and close-spaced hooping constitute proper reinforced concrete.

Another criticism that the writer would make concerns the floor-beams of Fig. 3. The main reinforcing rods are bent up at 45 degrees. Some of them run to the support, and some stop short of it; but they are not anchored over the supports. The beams are cluttered with stirrups, the use of which no one has ever been able to explain. If these main reinforcing rods had been run up over the top of the support and then curved down for anchorage, or, in the case of the middle post, had extended well into the next girder for anchorage, the construction would be very greatly improved. In the case of a common support of two girders, there is a large tension in the top of the

* *Journal*, Am. Concrete Inst., July, 1915, p. 397.

Mr. Godfrey.

girder over the supports. The rods of one girder should be bent up and continued well into the other one, in order to take this tension and anchor those rods for the diagonal tension or shear. In this bridge, a number of long rods could have been left projecting out of one floor-beam to be built into the other when the latter is made. Steel disposed in this manner is of definite value, whereas the aimless stirrups commonly used are a hit-and-miss sort of makeshift.

The steel used in stirrups would be vastly more beneficial if it was converted into main reinforcing rods curved up and anchored over the supports. Tests and common sense demonstrate this. Besides increasing the strength and safety of structures, this would make them cheaper and simpler to build.

Another detail of design to which the writer would call attention is on Plate XXXII. The sidewalk cantilever has a pull at the top flange which the writer calculates to be about 19 000 lb. To take this pull there are two rivets in tension on their heads, as shown by Plate XXX. The angle to which the sidewalk bracket is attached is connected to the bascule girder by only two rivets, and the top chord of the bracket is pulling directly on their heads. Besides this, the connection is eccentric. This is a detail that is very commonly slighted, and illustrates what the writer stated at the beginning of this discussion. A detail worse than this one could scarcely be devised. It behooves the engineers of this bridge to see to the correction of it before the sidewalk drops into the river.

The writer is familiar with a long viaduct where the sidewalk brackets are dependent on tension on four rivet heads the severance of which would mean, not only the dropping of the sidewalk, but the wrecking of the supporting girder.

Mr. Ritchie.

JAMES RITCHIE,* M. AM. SOC. C. E. (by letter).—The writer is much interested in this paper as he was in charge of the work for the firm of contractors which built all except the steel bascule spans. Mr. Chase has described very fully the methods used in the construction of the first half of the bridge, and a few additional details from the point of view of the contractor may be of interest.

Referring to the historical sketch, there are a few additional facts which came under the writer's observation. The first estimate of cost made by the designing engineers was $515 000, and on this basis the City of Toledo issued bonds and secured a fund of $525 000 for the building of the bridge. In the summer of 1907 proposals were asked for, but only one was received, which was more than $900 000. The matter was then discussed, and various plans were suggested, but finally, in July, 1909, proposals were again asked for, and three bids were received on the plans of the City and three alternate bids

* Cleveland, Ohio.

on the bidders' own plans. One of these alternate bids was for $897 000, and was made by the same firm which had bid alone on the first advertisement. One of the other bids was based on a different form of reinforcement of the arches from that shown on the City's plan, and the other bid was on a totally different method of construction. The lowest bid on the City's plan amounted to about $813 000, and therefore would require at least $300 000 more than the City had in its fund for the work. Even the lowest of the alternate plans required more than the City had appropriated, and the question which plan to adopt was discussed for several months. Finally, it was submitted to a board of expert engineers, and they recommended the adoption of the City's plan. The necessary additional funds were secured, and on March 3d, 1910, the contract was awarded. Immediately thereafter the firm which had submitted the lowest bid on an alternate design brought suit, through a resident of Toledo, to enjoin the City and the contractor from carrying out the contract. Decision on the suit was rendered in favor of the City and the contractor, in the Court of Common Pleas, and the case was appealed to the Circuit Court, which sustained the lower Court. Pending the decision in the Circuit Court, and at the request of the City, the contractor started work on the construction during June, 1910. Mr. Ritchie.

Plans were made by the writer which contemplated removing the old bridge to a temporary site and supporting the spans on pile piers, so as to leave the site of the new bridge unobstructed, but this idea was abandoned, not so much from fear of the action of ice on temporary work as because the recent decision of the Court had affirmed the fact that the method contemplated by the City's plans of building the superstructure in two halves was practicable, and the City did not wish to raise the question again before the public.

The easterly bascule pier, known as Pier IV, was placed so that the existing pier, supporting the east end of the 300-ft. swing bridge and the west end of a 175-ft. fixed span, had to be wholly removed and its foundation piles withdrawn before the bascule pier could be constructed. The coffer-dam for the bascule had to be at least 48 ft. 6 in. wide to contain the base of this pier, and this width had to be increased to 53 ft. on account of the projecting base of the old pier, which was at the extreme edge of the coffer-dam. In order to support the old bridge at this point without interfering with the river traffic, as well as the traffic over the bridge, it was necessary to have supporting trusses with a clear span of 60 ft., to provide very heavy pile supports for them, and to construct the trusses so that the old bridge would be supported rigidly for the fixed span and on the end wedges for the swing span. The temporary trusses had to sustain the loads at the site of the old pier, and this point was about 15 ft.

Mr. Ritchie.

from the west support and 45 ft. from the east support of the temporary trusses.

The old piers were first cut out by vertical slots of the width necessary to receive the temporary trusses, leaving a stone column to support the ends of the trusses of the old bridge. The temporary trusses were then placed in position and, after they were ready, the stone column was cut out for a sufficient space in which to place a jack on each side of the end post to hold it, while enough stone was removed to permit the placing of the wooden beams which were to transfer the load to the temporary trusses. This latter work had to be performed during the night when the traffic could be interrupted, and this was only from midnight until 5 A. M.

Four timber trusses were used at the bascule span and four plate girders at the east abutment. The top and bottom chords of the Howe trusses were each made of two pieces of 12 by 16-in. Oregon fir, each piece being 84 ft. long, thus being the full length of the trusses over the supports and requiring no splices, and thereby requiring only enough falsework to keep the sag out of the timbers while putting in the web members. The plate girders at the east abutment were put in position by a floating derrick, and also required no falsework for their erection.

The old bridge had a total width of 42 ft., and, in order to construct the north half of the new bridge, it was necessary to remove the north sidewalk of the old bridge. It was also necessary to move the two end spans into line with the other spans. These were originally placed so that there was a bend in the roadway on the bridge. To make this change in the end spans, falsework was built at the south ends of the piers, and, when all was ready, the two spans were jacked over to their new position on a Sunday afternoon.

While the first half of the bridge was being constructed, the City awarded a contract for a new foundation of a bridge at Ash Street, about a mile north of Cherry Street, contemplating the use of the old Cherry Street spans for the superstructure of the Ash Street Bridge, and as soon as traffic was turned over the first half of the new Cherry Street Bridge, the old spans were taken down, new floor-beams ordered, necessary repairs made to the old parts, and the spans erected on the Ash Street piers. The old trusses were found to be in good condition, and, being of wrought iron, the members had not deteriorated in the 30 years of use, having been kept well painted during all that period.

Mr. Chase has well said that the removal of the old bridge and the experience gained by the company in constructing the first half, was the reason for the rapidity with which the second half was completed. The company regrets that the plan of removing the old bridge to a temporary site—suggested in the first place—was not carried

out, as experience has shown that the cost of such removal would have been infinitesimal compared with the expense incurred and the delays occasioned by having to construct the bridge in two parts. The company's records show that the whole bridge could have been completed by the time that the first half was ready for traffic, thus saving more than a year of overhead expense, to say nothing of such difficulties as the obstruction caused by the piers, the temporary supports, the building of two coffer-dams, instead of one, for six of the piers, and the maintenance of traffic over the old bridge. Mr. Ritchie.

CLEMENT E. CHASE,* JUN. AM. SOC. C. E. (by letter).—The writer believes that Mr. Godfrey's apprehensions concerning the safety of the columns in the viaduct approach of the Cherry Street Bridge would be somewhat allayed if he were to see the completed structure, and thus get a better idea of its proportions and loads. The columns are 13 and 16 ft. long in the 2-ft. square and the 2 by 3-ft. sections, the cross-section in the remainder of the height being very largely increased. They were of square and oblong sections in the original design by the Osborn Engineering Company, and these shapes were retained in the revised design, prepared by Mr. Modjeski, largely for the sake of appearance and economic construction. Circular columns, although excellently shaped as far as strength alone is concerned, would have required expensive form work in this case, particularly at the fascia girder, and would not have corresponded with the architectural lines of the rest of the structure. The columns were of 1:2:4 concrete, very carefully mixed, and well worked in placing, to eliminate voids. Mr. Chase.

There may be doubt whether rectangular ties restrain effectively the rods in the middle of the side of a rectangular column, if it is stressed in compression so heavily that both steel and concrete are needed to carry the loads, but there is no reason why the restraint of the corner rods, even in such a case, should not be just as effective as is that of circular ties on the rods in round columns. In the moderately compressed columns at Cherry Street, the reinforcement, disposed as it was, is certainly capable of taking care of the small bending stress and any probable shock.

Some of Mr. Godfrey's other criticisms of the detailing in this structure have served to call attention to points which are incompletely or inaccurately presented in the drawings. The main rods of the floor-beams were bent up at an angle of 45° at both ends, and were anchored at one end by a loop into the fascia girder. At the other end they were extended as far out of the half of the girder first constructed as the adjacent old span would permit, which was sufficient to give an ample bond with the second half of the bridge. Also, all corners of

* Pittsburgh, Pa.

Mr. Chase.

columns and beams were chamfered liberally. These points are not shown accurately in the drawings.

The vertical stirrups, to which exception is taken, were spaced so that the shear on the concrete would not exceed 60 lb. per sq. in. Failure by diagonal cracking is resisted effectively by vertical stirrups, even if their exact disposition may not be subject to rigid analysis.

In commenting on the sidewalk bracket connection to the bascule girders, Mr. Godfrey has drawn his conclusions from insufficient information. Details of this connection are not given in the paper. In addition to the two rivets in tension, which are shown in the girder view, Plate XXXII—which Mr. Godfrey guessed were all that held the bracket up—there are from five to six rivets in shear, connecting a top horizontal plate to the top flange of the grider. The maximum stress in the top chord of the cantilever bracket is only 15 000 lb.

Mr. Ritchie has added several facts of interest which supplement the writer's account of the operations of his company. The construction of the bridge in two longitudinal halves certainly added greatly to the cost and length of time required to construct the complete bridge. The Cherry Street Bridge was, as he intimates, a storm center in Toledo's political life from its inception until its completion, and hard-won legal victories could not be jeopardized by changes in the plans of construction which otherwise might have been made as their desirability developed.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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Paper No. 1363

THE FAILURE AND RIGHTING OF A MILLION-BUSHEL GRAIN ELEVATOR*

By Alexander Allaire, M. Am. Soc. C. E.

With Discussion by Messrs. David Gutman, W. R. Phillips, and E. P. Goodrich.

Synopsis.

The object of this paper is to present to the attention of the members of the Society the history of a rather unusual engineering feat. The size of the building to be straightened, the angle to which it had tipped, and the weight to be handled, all combined to make the work unique.

The restoration of the elevator to a working condition may be divided into four parts:

1.—Making safe the foundations under the workhouse—a structure resembling a tall office building, resting on a very small base;

2.—Straightening the binhouse, a structure having an area of 15 000 sq. ft.;

3.—Providing this binhouse with a new and adequate foundation; and

4.—The renewal and repair of those portions of the original buildings which had been broken or deranged at the time of the failure.

This paper treats of the first three.

* Presented at the meeting of February 2d, 1916.

The Canadian Pacific Railway, which serves as the principal outlet from Western Canada, in 1911, found it imperative to provide relief for its Winnipeg Yards, which were yearly becoming less and less able to meet the demands on them. During October, November, and December, Canada's immense wheat-producing provinces of Manitoba, Saskatchewan, and Alberta ship their grain to the United States and Europe *via* the Canadian Pacific Railway to Fort William, where it is loaded on the Lake boats. To relieve the congestion, a cut-off was made, running approximately 3 miles north of Winnipeg, and one of the largest railroad gravity yards in the world was built at North Transcona, 7 miles northeast of the city. As an additional aid to the speedy shipment of the grain, a million-bushel elevator was constructed at the North Transcona Yard. This is to serve for storage during the "peak" periods, and relieve the car shortages by sending empties back West several days earlier than would be possible, should these cars have to go as far as Fort William.

The country in the vicinity of Winnipeg is flat, and the character of the soil is quite uniform throughout the district. First there is a stratum of about 2 ft. of heavy black loam, below which there is reddish gray clay, 5 or 6 ft. in thickness, and generally water-bearing, and this gradually changes to a blue clay extending to a depth of approximately 40 ft. below the surface, where it changes suddenly to white clay interspersed with limestone boulders. Underlying the white clay is found a stratum, averaging about 30 in. thick, of shattered limestone which in turn overlies the limestone rock. At Transcona this rock varied from 53 to 55 ft. below the prairie level.

Usually, the blue clay found at the depth from 7 to 8 ft. below the surface is very firm, and is capable of carrying a load of from 3 to 4 tons per sq. ft. As a result, it has been the practice in the Canadian Central West to use floating foundations. In building the elevator, therefore, this general custom was followed.

The ultimate loading of the clay under the mat was calculated at 3.3 tons per sq. ft. As a matter of precaution, soil-loading tests were made at different points on the site. From the result, it was thought that the customary slight initial settlement might be experienced, but that nothing more serious should occur.

The general layout consists of four units; a workhouse, 70 by 96 ft. and 180 ft. high; a binhouse, 77 by 195 ft. and 102 ft. high; a dryer-

house, 18 by 30 ft. and 60 ft. high; and a boiler-room equipped with two 100-h.p. locomotive boilers. The workhouse rests on a reinforced concrete floor, 30 in. thick, and is a reinforced concrete structure, with brick curtain-walls as the top is approached. The basement of the building is 16 ft. high. In it are the belts for transferring the grain from the cars to the conveyor-boots, and from the binhouse to the workhouse. The ground, or prairie-level, floor is occupied by the cleaning and drying machinery. Above these machines there are fifteen bins, 13 ft. in diameter and 70 ft. high, above which again are several floors carrying the weighing machines, etc. The floor, machinery, and wall loads above the second floor rest on twenty-four interior columns, placed in four rows of six columns each. The interior columns carry a load of about 800 tons each; the exterior ones carry about 500 tons each. Fig. 1 shows the workhouse in plan and elevation.

The binhouse, immediately north of the workhouse, consists of sixty-five circular bins arranged in five rows of thirteen each. These bins or tanks are 14 ft. 4 in. in diameter. The diamond-shaped spaces formed between the circular bins are also used for storage, each having a capacity of about 5 000 bushels. The bins, 92 ft. high, are surmounted by a cupola which houses the top conveyor and trippers, and extends the full length of the structure. With this the tanks are filled with grain. The bin walls are of concrete, 6 in. thick, with the customary reinforcement. Below the 12-in. reinforced concrete slab, which constitutes the bin bottoms, there are four conveyor-belt tunnels, 7 ft. wide, running the full length of the structure. These tunnels are formed by walls 16 in. thick and 7 ft. high, which rest on a 2-ft. mattress of concrete. Transverse to the main tunnel walls there are 15-in. cross-walls approximately 15 ft. from center to center. These are under the bin contacts. The tunnel or cross-walls were not bonded to the floor or bin slabs, and this constituted a considerable hazard during the straightening.

Excavation for the structure had been made to a depth of 12 ft. below the prairie level.

The dryer-house had broad footing courses under the walls. Although it was deemed advisable, later, to underpin this building, this presented no difficulty.

The Canadian Pacific Railway commenced storing grain in the binhouse in September, 1913. Considerable care was taken to regulate

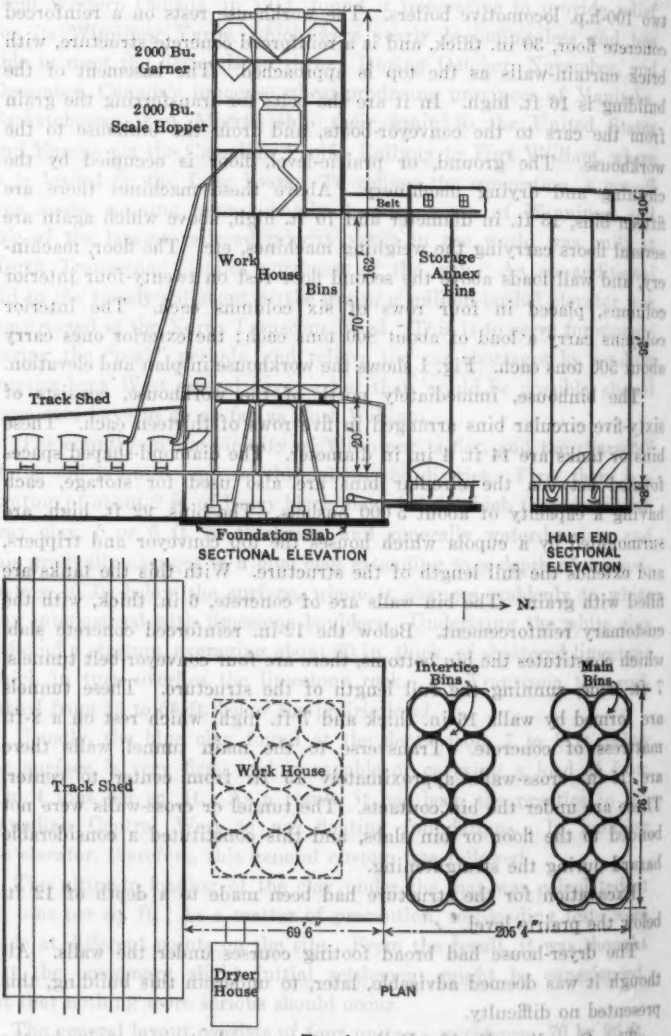

2000 Bu. Garner
2000 Bu. Scale Hopper
Belt
Work House Bins
Storage Annex Bins
Track Shed
Foundation Slab
SECTIONAL ELEVATION
HALF END SECTIONAL ELEVATION
N.
Intertice Bins
Main Bins
Work House
Track Shed
69' 6"
206' 4"
Dryer House
PLAN

FIG. 1.

the filling of the different tanks so as to distribute the load uniformly. Settlement began on October 18th. The bins at this time contained about 875 000 bushels of wheat. A vertical sinking of 1 ft. occurred within an hour after the settlement was first noted. This was followed by an inclination toward the west, which increased until, at the end of 24 hours, the binhouse rested at an angle of 26° 53′ from the vertical.

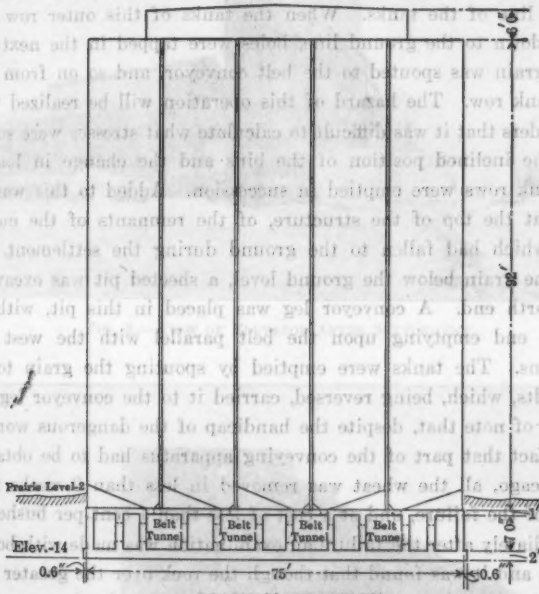


FIG. 2.

An examination showed the east side to be 5 ft. above, and the west side 29 ft. below, the original position; the building was also approximately 4 ft. lower at the north than at the south end. An upheaval of 5 or 6 ft. of the ground surrounding the bins occurred during the settlement. At an angle of 26° 53′ the center of gravity of the loaded structure had moved over almost to the low edge. The upheaval and the compacting of the soil along the west side saved it from completely falling over. Above this, on the west side, 52 ft.

of tank overhung the ground. This overhanging load, due to the grain-filled tanks, was considerable.

A careful examination was made of the structure and, remarkable as it may seem, practically no shear cracks were found. It was decided, therefore, that the first problem was to save the wheat. This was done by tapping the most westerly row of tanks at approximately the ground level and bleeding out the grain upon a belt conveyor parallel with the line of the tanks. When the tanks of this outer row were emptied down to the ground line, holes were tapped in the next row, and the grain was spouted to the belt conveyor, and so on from tank row to tank row. The hazard of this operation will be realized when one considers that it was difficult to calculate what stresses were set up, due to the inclined position of the bins and the change in loading as the tank rows were emptied in succession. Added to this was the menace, at the top of the structure, of the remnants of the cupola, part of which had fallen to the ground during the settlement. To remove the grain below the ground level, a sheeted pit was excavated at the north end. A conveyor leg was placed in this pit, with the discharge end emptying upon the belt parallel with the west side of the bins. The tanks were emptied by spouting the grain to the tunnel belts, which, being reversed, carried it to the conveyor leg. It is worthy of note that, despite the handicap of the dangerous working and the fact that part of the conveying apparatus had to be obtained from Chicago, all the wheat was removed in less than 3 weeks from the time of the failure, and at a cost of less than 1 cent per bushel.

Immediately after the failure an examination was made with boring machines, and it was found that though the rock over the greater area was at a depth of from 52 to 55 ft. below the prairie level, an unusual condition existed along about one-half of the length of the east side of the bins—a ridge of boulders some 12 ft. higher than the rock being encountered. This was the reason for the tipping over, for, as the initial vertical settlement took place, resistance was offered along the east side, probably through compacting the soil above the ridge of boulders. This produced a tendency of the building to cant to the west. The heavy load caused the clay to flow and the structure to settle.

Fortunately, the workhouse was only slightly affected by the movement of the binhouse. Only a few cracks showed in the north shed wall, no settlement taking place in the building proper.

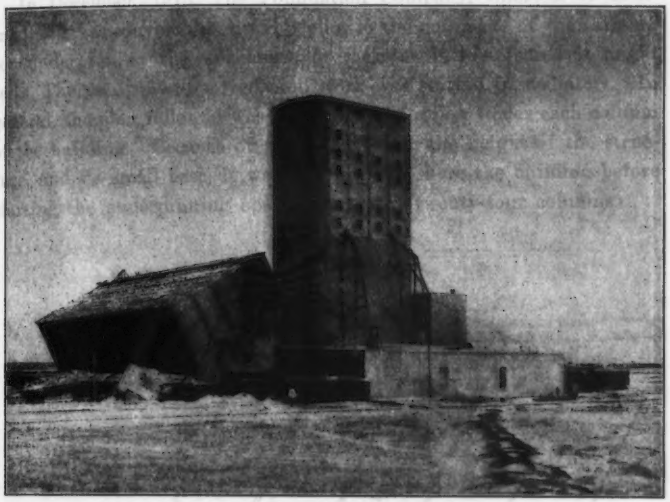


FIG. 3.—VIEW OF ELEVATOR AFTER SETTLEMENT.

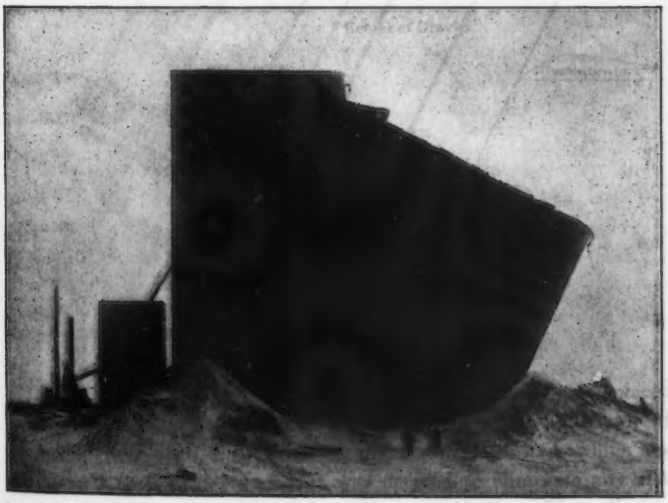
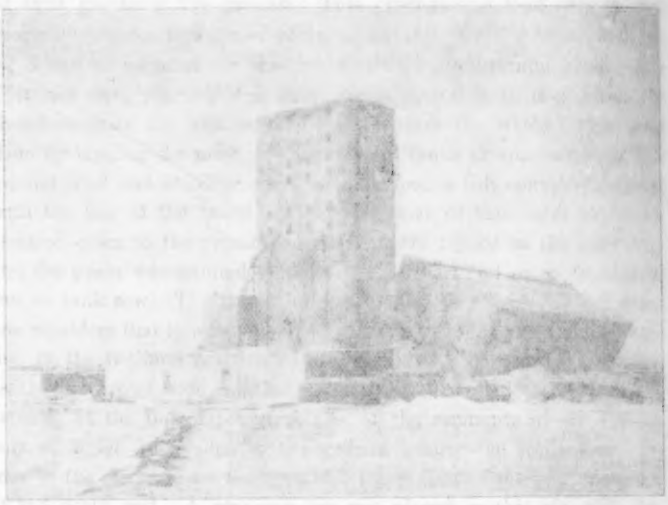
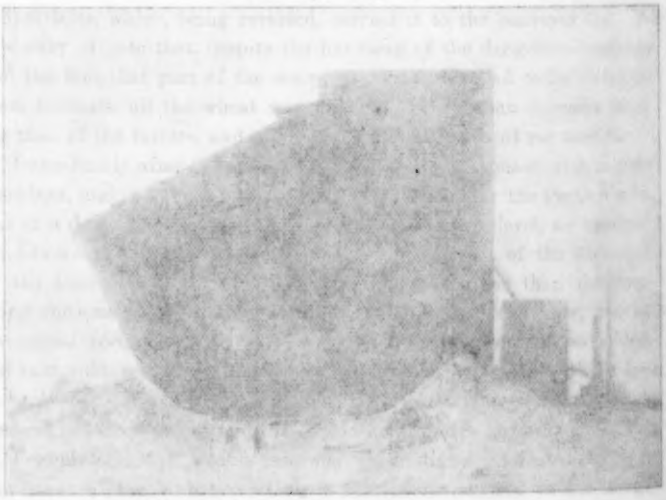


FIG. 4.—VIEW OF ELEVATOR AFTER SETTLEMENT.





In December, 1913, The Foundation Company, Limited, of Montreal and Vancouver, submitted a plan to the engineers of the railroad for underpinning the workhouse to rock, as it was feared it might fall. The plan was approved, and work was started immediately. In general, the plan followed was the sinking of a pier under each column of the building. Because of the heavy loads, the height of the structure, and its small base, it was necessary to shore the building before starting the underpinning operations at the twenty-four columns.

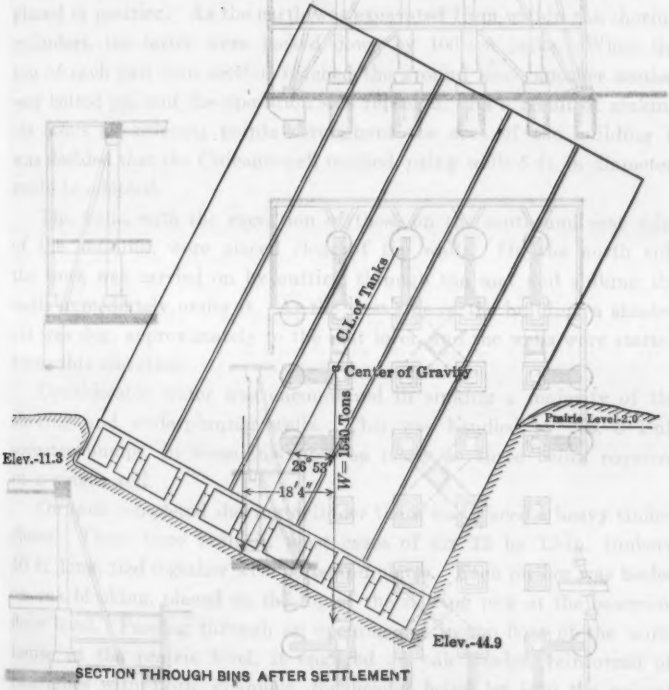


FIG. 5.

The shoring consisted of pushers from twenty small piers placed outside the building columns, as shown by Fig. 6. On account of the uncertainty of the condition of the material through which the piers would have to be carried, it was decided to eliminate any risk by sinking the first six with cast-iron shoring cylinders. The cylinders

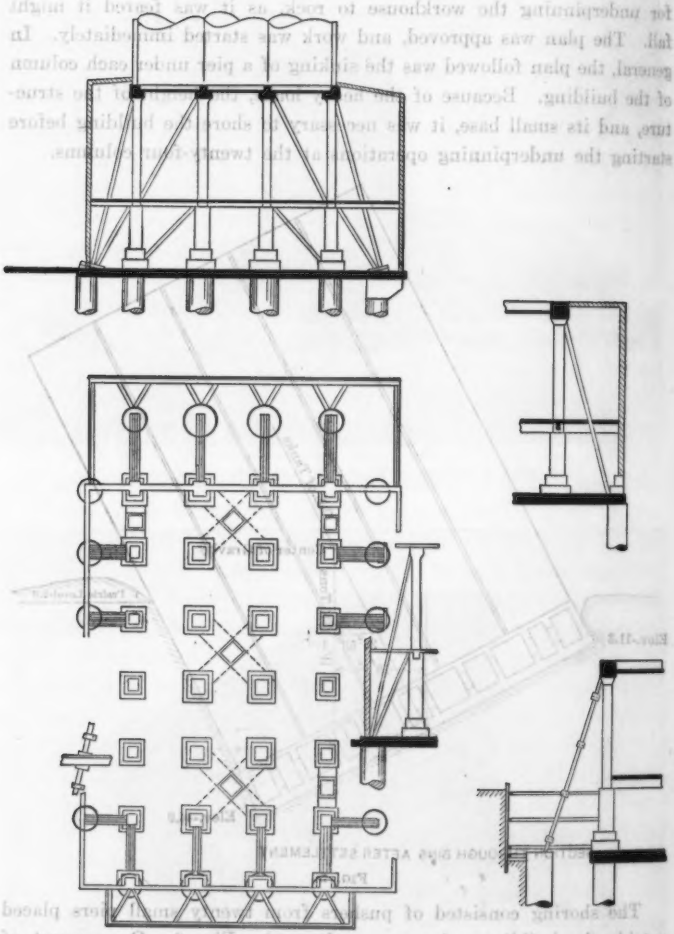


FIG. 6.

were 4 ft. in diameter, and built up in 3-ft. sections. Flanges were cast at each end of the sections for making connections. The one exception to this was at the bottom section, on which one flange was omitted, the unflanged rim being used as a cutting edge.

The shoring piers on the south and east sides were placed under the shed walls. Work on these was started by cutting a hole 4 ft. 6 in. square through the concrete floor, then drifting to the site of the pier. This excavation was sheeted, and the first section of the cylinders was placed in position. As the earth was excavated from within the shoring cylinders, the latter were jacked down by 100-ton jacks. When the top of each cast-iron section reached the ground level, another section was bolted on, and the operation was repeated. As a result of sinking six wells at different points throughout the area of the building it was decided that the Chicago well method, using wells 5 ft. in diameter, could be adopted.

The wells, with the exception of those on the south and east sides of the building, were placed clear of the walls. On the north side the work was carried on by cutting through the mat and sinking the wells immediately under it. At the west side of the building a sheeted pit was dug, approximately to the mat level, and the wells were started from this elevation.

Considerable water was encountered in sinking a majority of the shoring and underpinning wells. This was handled by No. 3 Pulsometer pumps, in some instances as many as three being required in a single well.

On each completed shoring cylinder there was placed a heavy timber shore. These were built in most cases of six 12 by 12-in. timbers, 40 ft. long, tied together with bolts and plates. Each pusher was heeled on oak blocking, placed on the top of the shoring pier at the basement floor level. Passing through an opening cut in the floor of the workhouse at the prairie level, it engaged an oak header, reinforced on two sides with 12-in. channels, the header being let into the column just below the bin floor by notching out the mushroom top, as shown by Fig. 7. Supplementing this was another pusher, extending from the shoring pier to the wall column at the first floor, and from this floor to the top of the interior column at the second-floor level.

Similar shores were placed on the shoring piers on all four sides of the building. These, combined with the columns, bin floor, and

reinforced mat, formed a truss arrangement by which the load of the building was largely transferred to the shoring piers. Later, while the interior piers were being sunk, timbers (shown by the dotted lines) were put in to take the load from the interior column bases.

All the shoring having been placed, the work of sinking the 5-ft. wells under the wall columns and the 6 ft. 6-in. ones under the interior columns was commenced. The columns are 15 ft. from center to center. To gain access to the under side of the exterior columns, holes 3 ft. 6 in. square were cut through the concrete floor mat between pairs of columns. Drifts 4 ft. wide and 6 ft. deep were driven in the direction of each column. As a well site was reached, the space was enlarged to a circular area slightly greater than the outside diameter of the well lagging. These drifts or tunnels were sheeted to prevent loss of ground, and afforded the only means of communication with the wells. The excavated material was hoisted in light galvanized-metal buckets which traveled on curved tracks or skidways, necessary on account of the center of the top of the underpinning well and opening in the floor being off center by 7½ ft. At the basement floor level the spoil was dumped into wheel-barrows, which were wheeled to a motor hoist, raised to the prairie level, and wasted at a short distance.

Prior to opening up the wells at the workhouse, levels had been taken on the column footings. As the piers were sunk, check levels were taken at regular intervals. As a further check on any movement, a plumb-bob weighing 280 lb. was suspended from the roof of the building down through an elevator shaft. The total distance was about 160 ft. The plumb-bob was immersed in a tank of water which was prevented from freezing by an electric coil. The position of the bob was noted every day.

Wells under the interior columns were approached in the same manner, except that the holes through the floor mat were cut centrally with respect to four columns, and handled the wells under them by drifting to each site in turn as one after another of the wells was completed. The distance from the center of the floor opening to the center of each underpinning pier in this instance was 10 ft. 6 in. Special curved skidways were used for these wells also. In all cases single-drum hoists, electrically operated, and having a rope speed of 130 ft. per min., were used to elevate the excavated material.

FIG. 7.—TYPICAL TOP OF WORKHOUSE PUSHER.

FIG. 8.—TRENCH, WITH BELT CONVEYOR, ALONG EAST SIDE OF BINHOUSE.

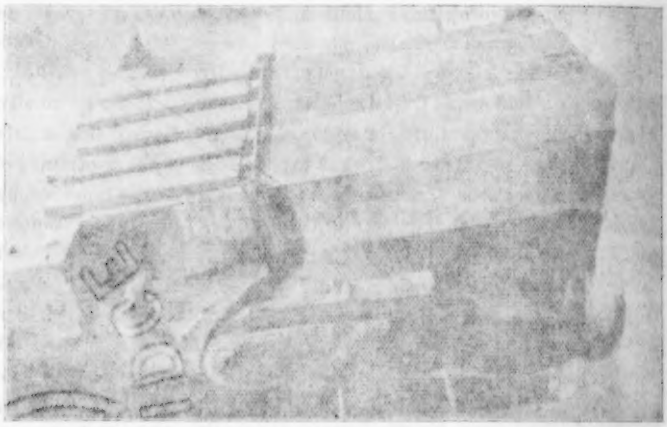


FIG. 1.—LANDING FOR AN AUTOMOBILE FERRY

FIG. 2.—TIMBER PIER UNDER CONSTRUCTION

Concrete, up to the top of the wells, was deposited through light 8-in. galvanized-iron pipe, cut to short lengths to accommodate them to the necessary curvature. Above this elevation, and up to within 8 in. of the under side of the columns, concrete was shoveled into place behind forms, the near side of which was built up as the concreting progressed. After this concrete had set sufficiently, the remaining 8 in. were finished both by grouting and by ramming in fairly dry concrete.

The workhouse operations were completed by about the beginning of June, 1914.

In the latter part of Feburary, 1914, permission was given to The Foundation Company, Limited, to proceed with the straightening of the binhouse. When the vertical position had been reached, it was planned to underpin it also by concrete piers to rock, these piers to be placed under the contact points of the tank walls in longitudinal rows. As a matter of economy, it was decided not to attempt to raise the building to its former elevation, but to straighten the structure by rotating it about the low edge; this was to be accomplished by excavating under the east or high side, and lowering it to the level of the low or west side. This meant that the mat in its final position would be approximately 38 ft. below the prairie level. As this was below the ground-water line, it was proposed to water-proof it.

The binhouse when empty of grain weighed 20 000 tons. Under the lower or west edge of the mat fourteen piers were sunk to rock and concreted. It was the intention to block the building off these piers to form a fulcrum, about which it would rotate as the excavating was done under the high side. The clay actually under the mat was removed in two ways: First, by working from under the high edge of the mat; and secondly, through holes in the mat. The former necessitated the excavation of a trench paralleling and flush with the east edge, as shown by Fig. 8. This was carried to a depth of about 8 ft. below the mat edge, was approximately 10 ft. wide at this point, and sloped back on the side away from the structure to the natural angle of repose of the soil. Drifts were driven from the trench toward the west under the mat. In the trench was placed a belt conveyor which discharged upon another belt, at the north end of the building, running at right angles to the first one. This second belt emptied into a hopper from which the earth was hauled by team

and spread over the prairie. The holes in the floor were in the compartments formed by the cross-walls between those of the main tunnels. The spoil passed through the holes and was handled by two conveyor belts, one in the first and the other in the third tunnel from the east. These belts discharged upon the transverse belt, previously mentioned as running along the north end. The gradual settlement of

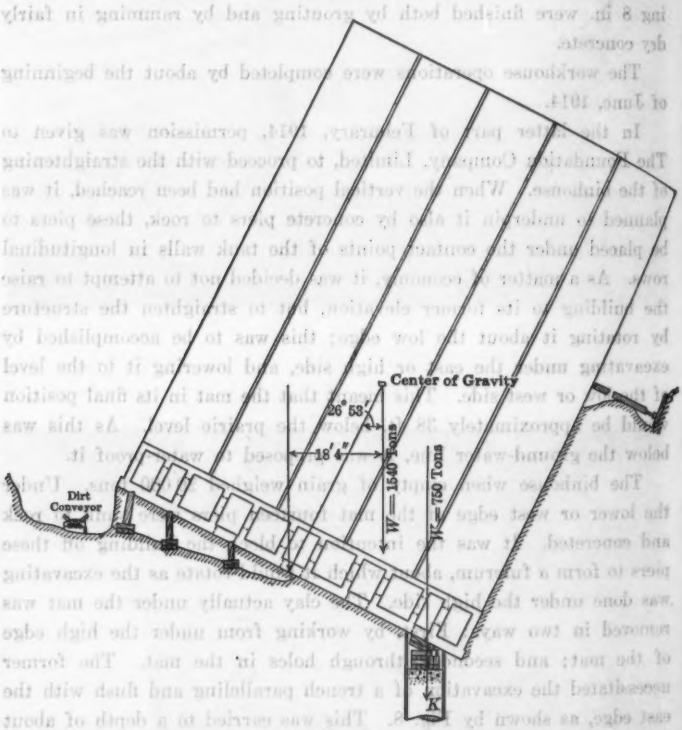


FIG. 9.

the high side was to be accomplished by weakening the core of earth left between the drifts. In driving the drifts, the earth wall between them was narrowed from the bottom up, the intention being that, as the building dropped, this core would offer increasing resistance to the movement, and that this movement could be controlled by removing the earth from the sides of the cores.

The fourteen piers under the low side, known as the *K* row, were placed on the longitudinal center line of the thirteen tanks, as shown by Fig. 9. They were placed under the contact walls and at the ends. Access to the sites of these wells was secured by cutting holes, 3 ft. 6 in. by 4 ft., through the mat in the compartments between the tunnels proper. The experience gained in sinking the workhouse wells showed that a large quantity of water would have to be handled in sinking the wells of the *K* row. Preparations for doing this were made by lagging down a large sump with 10 by 10-in. timbers at the north end of the building. This sump was carried to the depth of the low corner of the mat. It was placed inside the excavation which had been made to put in the grain leg and also the transverse belt conveyor used in connection with handling the spoil. In the sump was erected a duplicate set of electrically-driven centrifugal pumps and steam-driven piston pumps. Each set had a capacity of 1 200 gal. per min.

In sinking the wells of the *K* row, as well as the others described later, the work was performed under considerable difficulty, owing to the 27° inclination of the mat and the confined working space. Spoil from the different wells was raised by hand windlasses. The limited space and the small quantity of material to be handled would have made any mechanical arrangement more expensive.

The depth of sinking at the *K* wells was only 14 ft. Lagged wells of the Chicago type, 7 ft. in diameter, were used. The material was found to be very firmly compressed; in some instances the blue clay had been driven into the white stratum. The white clay, instead of being soft, as it had been found at the workhouse, was squeezed dry and hard, so that it was usually necessary to pick before shoveling into the buckets. No water was found until the shattered limestone stratum, immediately overlying the rock, was reached.

Not only was a large quantity of water found in the *K* wells, which from No. 1 to No. 6 came in at the rate of 1 150 gal. per min. in each well, but, in addition there was the danger that the level to which it would normally rise was approximately 14 ft. above the mat at the openings. This made imperative the duplicate pumping system at the sump. The insufficiency of room further aggravated the water troubles.

To do away with the multiplicity of piping of the small steam units and the almost unbearable heat from them, belt-driven, electrically-operated centrifugal pumps were erected immediately above the wells. These were placed on the main tunnel floors, and independent holes for the pump suctions were cut through the mat. As it was considered inadvisable to open up too many wells at one time, alternate ones only were started. As the concreting of these piers was finished, oak cribs were placed upon them to take the load of the building, as shown by Fig. 9.

As the work on the *K* piers was nearing completion and the majority of them had been blocked up, the construction of the remaining 56 piers was commenced, it being the intention to sink these piers while the righting process was under way. These were opened up as far as possible in widely scattered positions. Although quantities of water were encountered, particularly at the north end of the building, the working conditions were somewhat better than at the *K* row, and the work progressed steadily. These wells were also 7 ft. in diameter; they were placed under the contact walls and tank ends, and were in all cases carried to rock. They were arranged in four rows, starting from the east, and designated as *G*, *H*, *I*, and *J*.

At about this time the original plans were changed, owing to the fact that the railroad engineers had decided that they would prefer to have the tunnel floors above the ground-water level. To accomplish this the new plan contemplated allowing the bins to rotate about the *K* row until the angle, 16° 30′, had been reached, at which time the binhouse was to be pivoted about the *I* row of piers to an angle of 8° 30′. At this point the structure was to be again pivoted about the *H* row of piers into a vertical position.

As the drifts or tunnels were driven under the high side of the mat, 12 by 12-in. posts were set up in them on mudsills of such an area that the load imposed on the soil was about 3 tons per sq. ft. See Fig. 10. These helped to make up for that resistance against the mat which was lost as the drifts advanced. Later, as the excavation was extended so that the sinking of the wells in the *G*, *H*, *I*, and *J* rows could be proceeded with, more posts on mudsills were added, for the same reason.

The number of posts put in also enabled the settlement of the structure to be closely regulated. This was preferable to depending

FIG. 10.—EXCAVATION TOWARD WEST FROM UNDER EAST EDGE OF MAT.

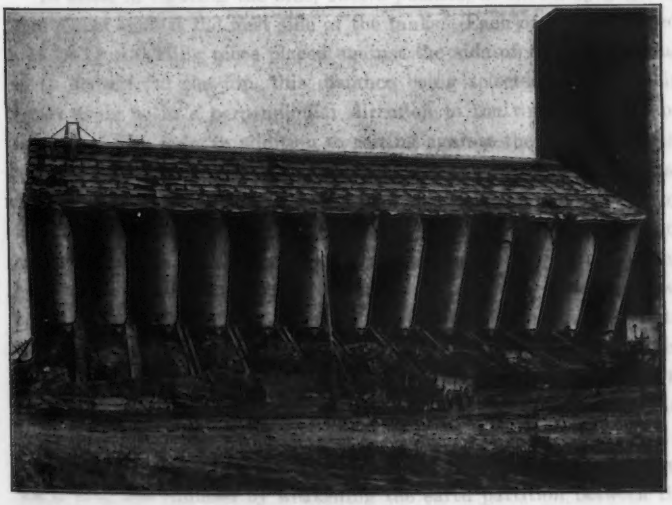
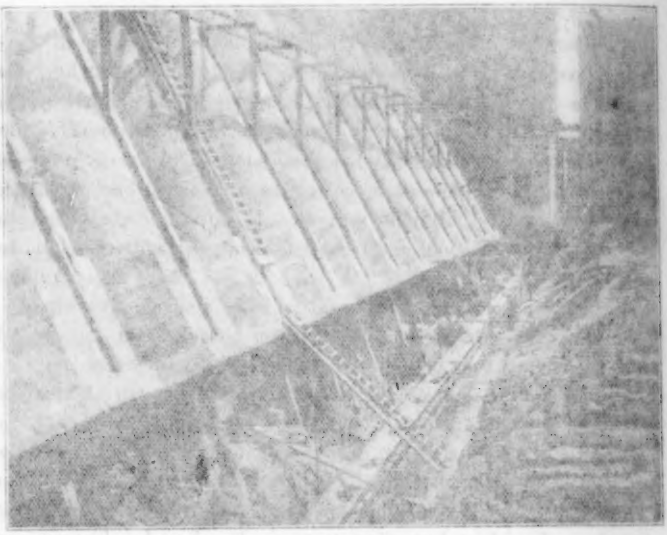
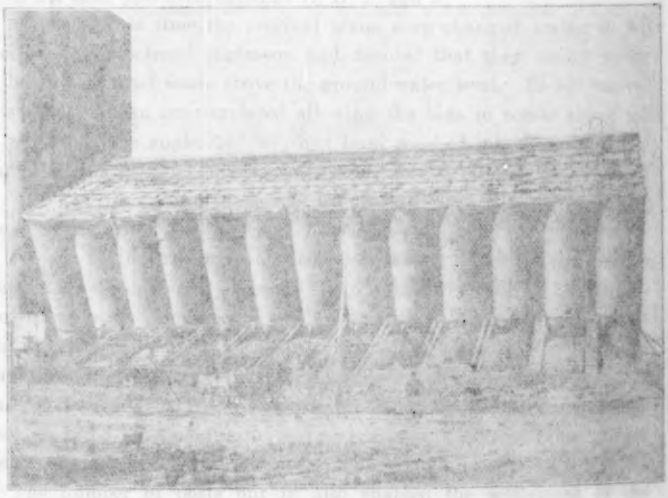


FIG. 11.—LINE OF PUSHERS AGAINST WEST SIDE OF BIN STRUCTURE, SHOWING HOLES THROUGH WHICH GRAIN WAS TAKEN OUT.





only on the clay ribs, which, on occasions, had a tendency to break off in large masses. This fracturing of the clay cores also necessitated increasing the number of posts.

The posts as well as the shoring screws, mentioned later as being placed on the *G* and *H* piers, were also necessary on account of the fact that the reinforced mat, tunnel walls, and bins were not tied together vertically in any manner. Without some such precaution the mat could have broken off, or the three parts could have slid on one another.

With the bins at rest at the angle of 26° 53′, calculations showed that the loads at the pier sites were as follows:

Those in	*G*	row.....	Uplift
" "	*H*	"	11 tons per pier.
" "	*I*	" 227	" " "
" "	*J*	" 529	" " "
" "	*K*	" 773	" " "
Total per transverse row....1 540 tons.			

To assist in righting the bins, twelve pushers, as shown by Fig. 11, were placed against the west side of the tanks. Each of these engaged a 12 by 12-in. waling piece placed against the side of the tanks about 45 ft. down from the top, this distance being selected so that their forces would be in a perpendicular direction to the vertical height of the tanks. The wale, in addition to resting against the thirteen tanks, was also posted into the contact walls, for it was at these points that the pushers were applied. Each pusher was composed of two 12 by 12-in. timbers, spaced 12 in. apart by using spreaders of that dimension, and tied together by plates and bolts. Each was 60 ft. long, and each had two screws, one in each of the 12 by 12-in. timbers, these screws heeling against ample timber mats. The screws were operated until the mat had reached an angle of 8° 30′. The line of action of the pushers was lowered once during the righting operation so as to maintain as nearly as possible their perpendicular direction against the sides of the tanks.

The initial righting movement, namely, that of rotating about the *K* row, was induced by weakening the earth partition between the drifts. Practically, a continuous daily movement of from 3 to 4 in.,

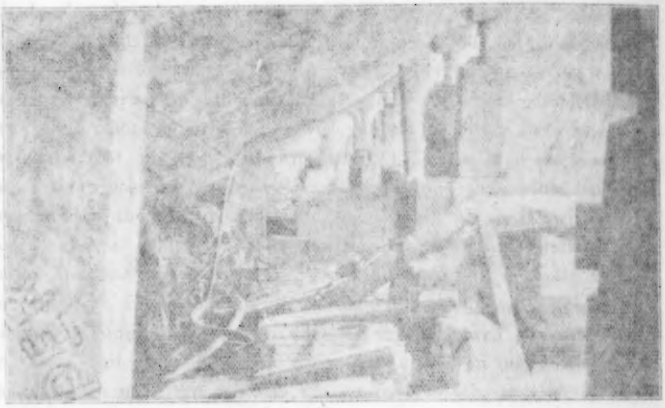
at the eastern edge, was maintained. When the bin floor had reached an angle of 18°, the movement was stopped for the purpose of placing the hardwood rockers at the *I* row of piers.

During this initial movement all the wells under the mat were successfully bottomed and concreted to the heights required for carrying on the work. As rapidly as the *G* and *H* rows were completed, cribwork was placed on each pier, with 40-ton shoring screws in contact with the mat, as shown by Fig. 12. These screws made possible a much more positive control in lowering the mat than could have been attained by the clay rib alone. The placing of the shoring screws and posts on mudsills also permitted the removal of all the clay under the mat back to the west side of the *H* row of wells, approximately 23 ft. from the edge of the mat. The removal of this excavation from under the eastern edge of the mat was thus carried on more economically than if the excavation had been made through the holes in the mat. In fact, as the lowering of the mat continued, a uniform depth of the clay of approximately 8 ft. from the outer edge and 4 ft. at the line back of the *H* row was maintained—the belt conveyor used for the disposal of the material being also lowered from time to time as the general excavation progressed.

Shortly after starting to rotate the structure about the *K* row, readings showed that a lateral or sliding movement to the eastward had set up. This was hard to understand, on account of the inclination of the bins to the west; also because of the compressed clay fronting the under side of the mat, and the fact that the dead load of the structure was 20 000 tons. Notwithstanding this, however, the tendency was always in evidence until the structure was pivoted on the *I* row of piers. To resist the movement, 12 by 12-in. timber kicking braces (Fig. 13), usually arranged in pairs, were used. These were of two lengths: short ones engaging the high edge of the mat, and long ones reaching in under the mat to notches cut in its under side, as shown by Fig. 14. In both cases they heeled against the earth embankment on the far side of the trench, where liberal timber heels had been set up. To retard the lateral movement still more, short inclined posts were set up on the *K* piers, their upper ends engaging notches cut in the mat. In addition, approximately 3 000 cu. yd. of earth, pressing against the bins along the west side, were removed. This total lateral movement to the west finally amounted to 3 ft. 3 in.

FIG. 12.—SHORING SCREWS, ON "G" ROW OF PIERS, CARRYING EDGE OF MAT.

FIG. 13.—KICKING BRACES AGAINST EASTERN EDGE OF MAT AT BINHOUSE.



On August 13th the structure was brought to rest at an angle of 18° by using cribbing placed on the *J* row of piers. There were two of these cribs on each pier, and they were placed in a direction transverse to the longitudinal axis of the bins. They had a total area in plan of 4 by 7 ft. The rear one was 2 by 4 ft., and was removed later to give room for the shoring screws; the former one was used for following up with blocking as the structure rotated about the *I* and *H* rows of piers.

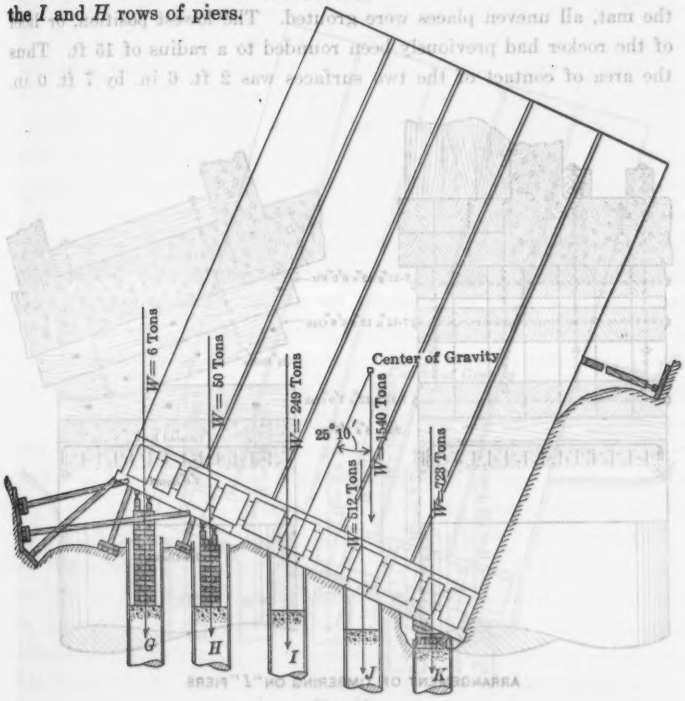
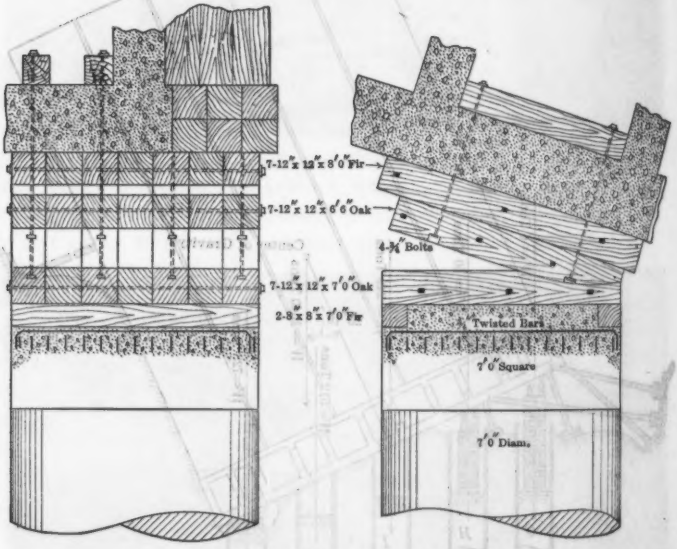


FIG. 14.

To form convenient working places, all the tops of the piers were made 7 ft. square. In preparing the piers for the rockers, as shown by Fig. 15, 8 by 8-in. timbers were embedded in the edges parallel to the longitudinal axis of the bins. The rockers consisted of two parts: the shoe and the rocker proper. The former was made by covering the top of each pier with 12 by 12-in. oak timbers, laid in

the direction of the movement and strongly bolted together. The upper surface of this shoe was concaved to a radius of 30 ft., and also in the direction of the rotation.

The rockers consisted of two courses of 12 by 12-in. oak timber, seven pieces in each course, directly above the timbers in the shoe. These two courses were strongly bolted together and also to the mat. To make certain of an even bearing against the rough bottom of the mat, all uneven places were grouted. The lowest position, or heel of the rocker had previously been rounded to a radius of 15 ft. Thus the area of contact of the two surfaces was 2 ft. 6 in. by 7 ft. 0 in.



ARRANGEMENT OF TIMBERING ON "I" PIERS

FIG. 15.

Clearance was allowed between the rockers and the shoe, so that contact was not made until the bins reached an angle of 16° 30'. The allowance was made in order to provide for the possibility of a slight settlement during the time required to set up the rockers. When the mat had reached this angle, the center of gravity had shifted to a position 11 ft. 3 in. west of the center of the structure. At the angle

of 26° 53′, it had been 18 ft. 4 in. to the west. In the new position the figured pressures on the piers (see Fig. 16) were as follows:

G.....	80 tons.
H.....	193 "
I.....	316 "
J.....	423 "
K.....	528 "

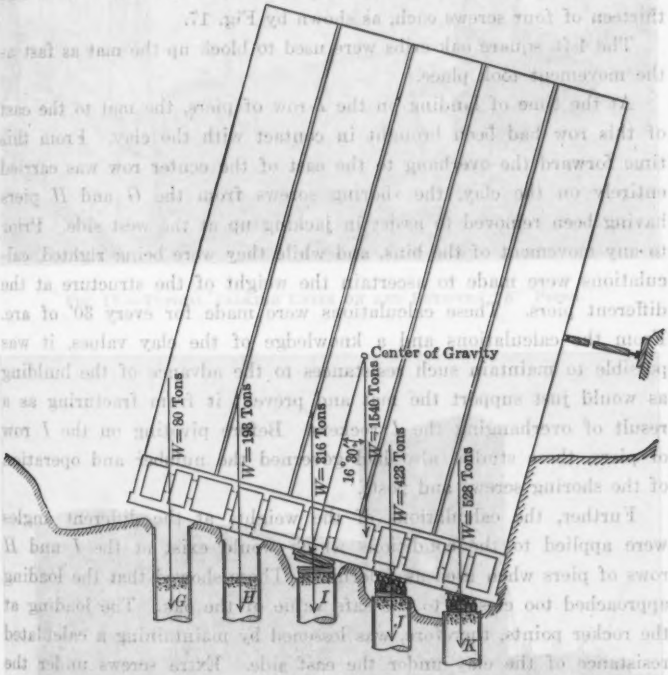


FIG. 16.

The rockers on the *I* row having been completed, the next step consisted in preparing the *K* and *J* rows for the shoring screws. Excavation to a depth to give ample room was made from the east face of the *J* piers to the west face of the *K* piers. This excavation stepped up as the *J* piers were approached, so as to maintain a practically constant head room and reduce the quantity of back-fill required later.

Between the piers of the *K* row the excavation exposed the white clay, previously mentioned as having been very much compacted. Over the clay and extending from pier to pier throughout the total length of the *K* row was laid 1 ft. of concrete. On these slabs were placed oak timbers, and on the latter four 50-ton shoring screws were set up. Twelve 50-ton screws, surrounding oak cribs 4 ft. square, were placed on the *K* and *J* piers. At the time of starting the movement there were, therefore, twenty-eight jacking units of twelve screws each and thirteen of four screws each, as shown by Fig. 17.

The 4-ft. square oak cribs were used to block up the mat as fast as the movement took place.

At the time of landing on the *I* row of piers, the mat to the east of this row had been brought in contact with the clay. From this time forward the overhang to the east of the center row was carried entirely on the clay, the shoring screws from the *G* and *H* piers having been removed to assist in jacking up at the west side. Prior to any movement of the bins, and while they were being righted, calculations were made to ascertain the weight of the structure at the different piers. These calculations were made for every 30′ of arc. From the calculations and a knowledge of the clay values, it was possible to maintain such resistances to the advance of the building as would just support the mat and prevent it from fracturing as a result of overhanging the *I* rockers. Before pivoting on the *I* row of piers, these studies also had governed the number and operation of the shoring screws and posts.

Further, the calculations of the weights at the different angles were applied to the conditions which would exist at the *I* and *H* rows of piers when used as fulcrums. These showed that the loading approached too closely to the safe value of the oak. The loading at the rocker points, therefore, was lessened by maintaining a calculated resistance of the clay under the east side. Extra screws under the low side were used to overcome this resistance.

The righting operations were continued, both by lessening the resistance of the clay under the overhanging portion of the mat and by jacking up the low side. Trenches were excavated under the mat from the *H* rows of piers eastward. These trenches extended to within about 3½ ft. of the east edge of the mat, and averaged 4 ft. in depth, except when the mat was approaching the horizontal position, when

FIG. 17.—TYPICAL JACKING UNITS ON AND BETWEEN "K" PIERS.

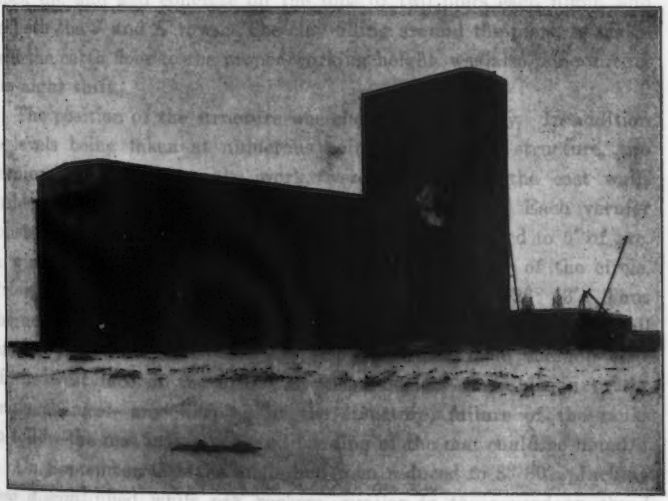
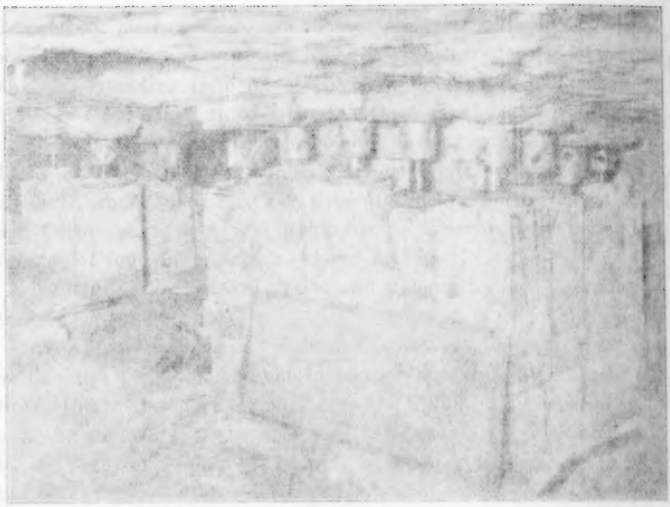
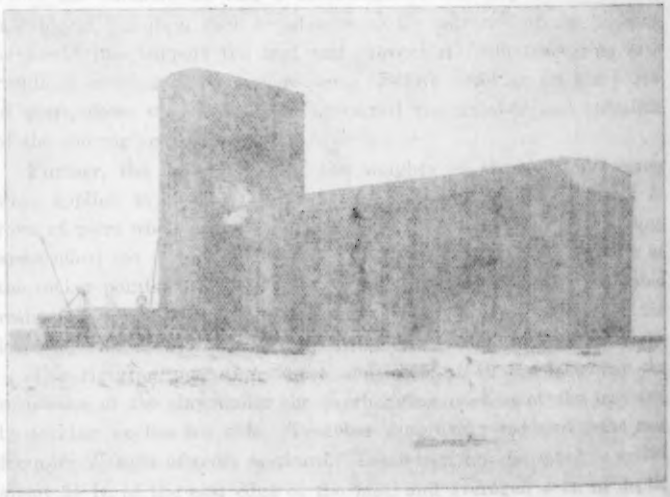


FIG. 18.—STORAGE BINS IN RIGHTED POSITION.





they were allowed to fill. The bank of clay between the *H* and *I* rows was left in place. The width at the bottom of the trenches averaged about 3 ft., the sides being sloped so that each rib between two trenches had a triangular section. Continuous weakening of the clay ribs and deepening of the trenches was maintained at a rate practically equivalent to the uplifting thrust of the shoring screws. Spoil from the trenches was handled through holes in the mat.

The jacking part of the lifting operation was done by gangs made up of three men. Each gang was supplied with a 6-ft. steel bar 1⅛ in. in diameter, which, when inserted in the head of the shoring screw with three men pulling on it, was equivalent to an effort of approximately 55 tons. In the *K* row each gang handled eight screws, and six was the allotment for the *J* row. The periods of work and rest were carefully arranged, and a uniform application on the screws throughout the total length of the bins was maintained during such periods. The jacking-up process was only carried on during the day shifts. At the same time, six gangs of two men each were kept busy fleeting the screws and following up with oak blockings as progress was made. The night shift was utilized to fleet the screws, block up, and add concrete on the tops of two piers each night, and on both the *J* and *K* rows. The clay filling around the piers, to maintain the earth floor to the proper working height, was also done during the night shift.

The position of the structure was checked twice daily. In addition to levels being taken at numerous points around the structure, two verniers, fabricated on the work, were attached to the east walls inside of the north and south tanks of the *G* row. Each vernier consisted of an arc of a circle of 15 ft. radius, graduated to 5′ of arc. A wire with weighted lower end hung from the center of the circle. Prior to any movement of the bins, this had been set to 26° 53′. Thus the actual angle of the structure could be ascertained quickly at all times. A 240-lb. plumb-bob was also suspended from the top edge of the west face of the bins. By these devices—in addition to the angle changes—any warping of the structure, failure of the tanks to follow the mat movement, and bending of the mat could be noted.

On September 17th the angle had been reduced to 8° 30′. Jacking was discontinued while oak rockers, similar to those used on the *I* piers were placed on the *H* piers. As in the previous case, at the *I*

piers, allowance had been made for the creeping of the structure during the time it took to place the rockers, and they did not come to a full bearing until the angle 7° 30′ had been reached. The shoring screws were now removed from the pushers at the west side of the building and set up on the *I* piers. The grouping of screws during this, the final movement, was as follows: *K* piers, each twelve screws; *J* piers, each ten screws; and *I* piers, each eight screws. At this angle (see

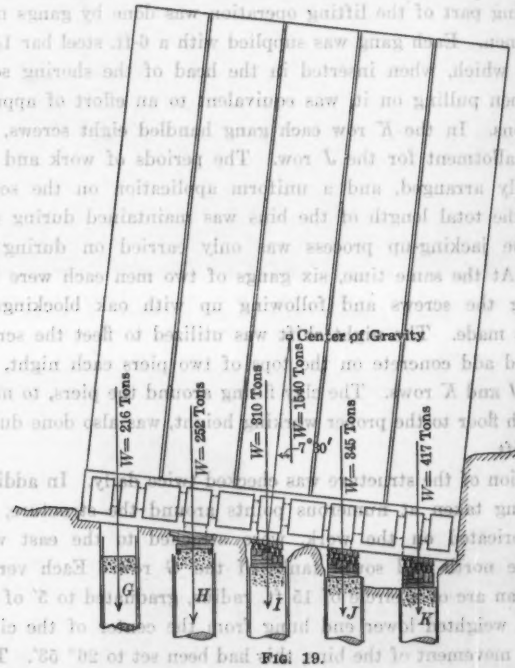


FIG. 19.

Fig. 19) the pier loading was as follows: *G*, 216 tons; *H*, 252 tons; *I*, 310 tons; *J*, 345 tons; and *K*, 417 tons. The center of gravity, in the meantime, had shifted until at this angle it was only 3 ft. 3 in. west of the center line of the structure. During this last operation it was found advisable to assist the screws on the *I* piers by using oak wedges, 4 ft. long, 6 in. wide, and 3 in. thick at the butt end; these were driven between the shoe and the rocker on these piers. This was made necessary on account of the tendency of the mat to sag.

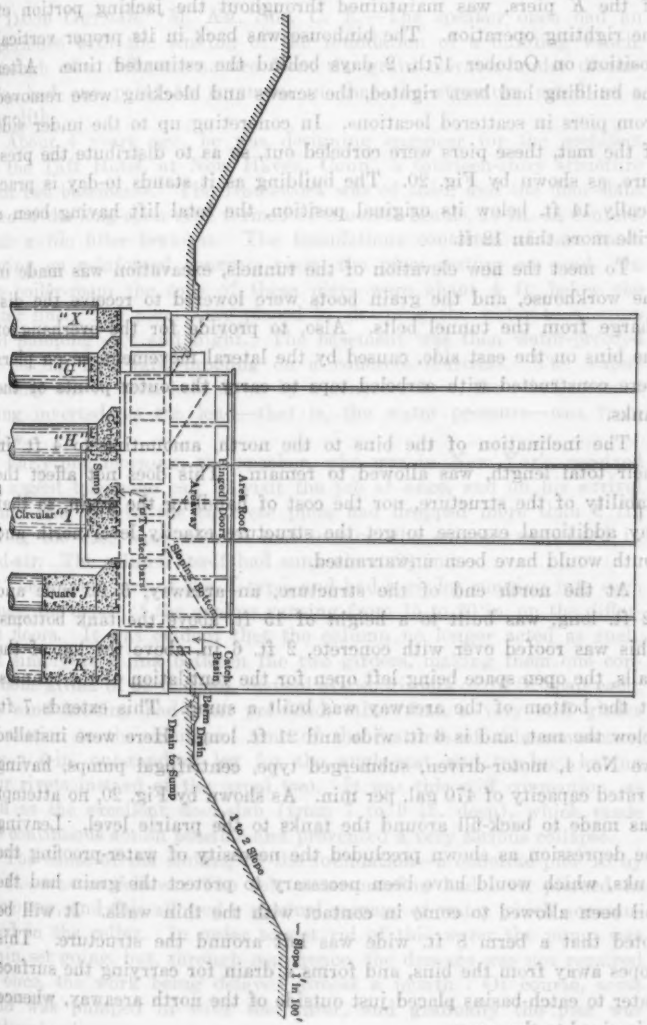

"X"
"G"
"H"
Circular "I"
Square "J"
"K"
Sump
Arch Roof
Hinged Doors
Archway
Square Bottom
Catch Basin
Drain to Sump
1 to 2 Slope
Slope 1 in 100'

FIG. 20.

An average rate of 4 in. in 10 hours, measured on the center line of the *K* piers, was maintained throughout the jacking portion of the righting operation. The binhouse was back in its proper vertical position on October 17th, 2 days behind the estimated time. After the building had been righted, the screws and blocking were removed from piers in scattered locations. In concreting up to the under side of the mat, these piers were corbeled out, so as to distribute the pressure, as shown by Fig. 20. The building as it stands to-day is practically 14 ft. below its original position, the total lift having been a trifle more than 12 ft.

To meet the new elevation of the tunnels, excavation was made in the workhouse, and the grain boots were lowered to receive the discharge from the tunnel belts. Also, to provide for the overhang of the bins on the east side, caused by the lateral movement, seven piers were constructed with corbeled tops to carry the outer points of the tanks.

The inclination of the bins to the north, amounting to 4 ft. in their total length, was allowed to remain. This does not affect the stability of the structure, nor the cost of handling the grain, so that any additional expense to get the structure exactly level north and south would have been unwarranted.

At the north end of the structure, an areaway, 6 ft. wide and 52 ft. long, was built to a height of 15 ft. above the tank bottoms. This was roofed over with concrete, 2 ft. 6 in. above the top of the walls, the open space being left open for the ventilation of the tunnels. At the bottom of the areaway was built a sump. This extends 7 ft. below the mat, and is 6 ft. wide and 21 ft. long. Here were installed two No. 4, motor-driven, submerged type, centrifugal pumps, having a rated capacity of 470 gal. per min. As shown by Fig. 20, no attempt was made to back-fill around the tanks to the prairie level. Leaving the depression as shown precluded the necessity of water-proofing the tanks, which would have been necessary to protect the grain had the soil been allowed to come in contact with the thin walls. It will be noted that a berm 8 ft. wide was left around the structure. This slopes away from the bins, and forms a drain for carrying the surface water to catch-basins placed just outside of the north areaway, whence it is piped to the sump.

DISCUSSION

Mr. Gutman.

DAVID GUTMAN,* M. AM. SOC. C. E.—The speaker once had an experience with the sinking of the foundation of a building which, although of a different nature from the grain elevator under discussion, had one point in common, in that the structure acted as a monolith.

About 4 years ago, he was designing engineer for the architect on the Taft Hotel, at New Haven, Conn., a fourteen-story structure with two basements. The framework was of steel, and the floor-slabs were of the long-span type formed of concrete beams, 16 in. on centers, with a tile filler between. The foundations consisted of cast bases resting on reinforced concrete piers, the piers resting on sand. In the boiler-room the tops of these piers were about 4 ft. below the water line. The piers were placed by draining the water to a sump and pumping day and night. The basement was then water-proofed by placing felt water-proofing on a concrete mattress. The water-proofing was in turn held down by a concrete floor, the reinforcement being inverted, as the load—that is, the water pressure—was from below.

Early one morning, the speaker, who was in New York, received an urgent telephone call to visit the job at once, and on his arrival found that one of the concrete piers had dropped more than 2 ft., taking the cast base with it and leaving the column suspended in mid-air. The column itself had sunk about 2½ in.

The column was of the I-type, and had a girder running into each flange, the depth of the girders varying from 15 to 20 in. on the different floors. It was evident that the column no longer acted as such, but simply as a link between the two girders, making them one continuous girder of 32 ft. span, instead of two girders of 16 ft. span each. No wind bracing had been provided, other than a very stiff girder connection to the columns. One of the features of this connection was a 6-in. outstanding leg for the angle-seat and top-lug, having four rivets instead of the usual two. It was this stiff connection, as well as the excellent floor-slab (from 7 to 9 in. deep), which made the continuous action possible and prevented a very serious collapse.

The cause of the sinking of the foundation—a concrete pier 10 by 10 ft.—was as follows: The felt water-proofing had been pierced in some way, and this allowed a gradual seepage of water which accumulated in the cellar. In order to get rid of this water the pump was again set going, but, through negligence, the damage was not repaired at once, the work being delayed almost a month. Of course, some sand was pumped in with the water, and gradually the pier was undermined.

* Mount Vernon, N. Y.

Mr. Gutman. Two heavy plate girders were placed on opposite sides of the column, spanning to the two good piers on each side. These girders were then riveted to the column, and jacks were placed under the ends. By these jacks, the column was pushed up about ¾ in. It was not considered wise to raise it the whole amount of the settlement—2½ in.—as this would have cracked the plaster on the upper floors, most of which had already been placed. The piers on each side showed no settlement whatsoever, either before or after the jacking.

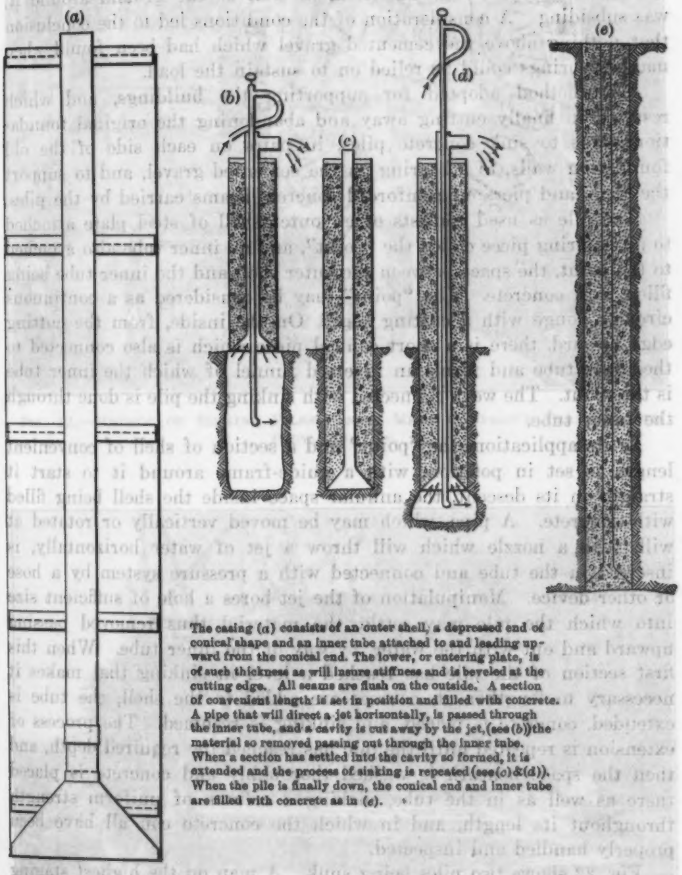
Mr. Phillips. W. R. Phillips,* M. Am. Soc. C. E. (by letter).—A brief description of some work brought about through the failure of the foundations at the Union Meat Company's plant may be of interest in connection with the discussion of this paper. The cases were to some extent similar, although the manner of sinking the new foundations was different. Unlike the work at Winnipeg, this was handled entirely from the surface, and was independent of underground conditions, such as water, depth, or material, so long as the latter could be cut by the jet.

The plant of the Union Meat Company is housed in a group of buildings suited to the packing business, and is on the south shore of Oregon Slough, Columbia River, at North Portland, Ore. The principal buildings of the group are the main packing house and the tank house. The former is 122 by 166 ft., and, in its highest part, has seven floors. The latter is 33 by 52 ft., and has four floors.

As designed originally the buildings were to rest on concrete foundations of considerable depth, supported by wooden piles. The piles were properly driven and, after the foundations were built, the spaces between and around the walls were filled with sand dredged from the river. In preparation for the work, wash borings had been made to a depth of 116 ft. below the surface of the ground, where cemented gravel was found. The borings produced nothing to indicate that the ground would not remain stable under the load to which it was to be subjected, and, in this case at least, may be considered as an illustration of unreliability in a test of this class, as the condition on which the ground depended for its stability was not such as would be readily discovered by wash borings. It was not until the concrete piles for the new foundation were being sunk that the clue was obtained whereby the condition that might be responsible for the subsidence was discovered. During that process a horizontal stratum of dense material about 6 ft. thick, was found at a depth of about 65 ft. below the surface, and, as it happened, just beneath the ends of the wooden piles, which aroused suspicion that the water beneath it was sustaining the ground, and that the water was being slowly displaced by the added load. The correctness of this theory was seemingly demonstrated by sinking

* Portland, Ore.

Mr. Phillips.



The casing (*a*) consists of an outer shell, a depressed end of conical shape and an inner tube attached to and leading upward from the conical end. The lower, or entering plate, is of such thickness as will insure stiffness and is beveled at the cutting edge. All seams are flush on the outside. A section of convenient length is set in position and filled with concrete. A pipe that will direct a jet horizontally, is passed through the inner tube, and a cavity is cut away by the jet, (see (*b*)) the material so removed passing out through the inner tube. When a section has settled into the cavity so formed, it is extended and the process of sinking is repeated (see (*c*) & (*d*)). When the pile is finally down, the conical end and inner tube are filled with concrete as in (*e*).

FIG. 21.

Mr. Phillips.

a pipe through the dense material. In this pipe water was found to stand at a height of 10 ft. above that of the adjacent slough.

Not until sand was dredged in to fill around the foundation was there evidence of settlement, but with that to start it, it was soon demonstrated that the foundation, as well as the ground around it, was subsiding. A consideration of the conditions led to the conclusion that nothing above the cemented gravel which had been found when making borings could be relied on to sustain the load.

The method adopted for supporting the buildings, and which resulted in finally cutting away and abandoning the original foundations, was to sink concrete piles, in pairs on each side of the old foundation walls, to a bearing on the cemented gravel, and to support the walls and piers on reinforced concrete beams carried by the piles.

The pile as used consists of an outer shell of steel plate attached to an entering piece called the "point", and an inner tube also attached to the point, the space between the outer shell and the inner tube being filled with concrete. The "point" may be considered as a continuous circular gouge with a cutting edge. On the inside, from the cutting edge upward, there is a short conical piece which is also connected to the inner tube and forms an inverted funnel of which the inner tube is the spout. The work connected with sinking the pile is done through the inner tube.

In its application, the "point" and a section of shell of convenient length is set in position, with a guide-frame around it to start it straight in its descent, the annular space inside the shell being filled with concrete. A pipe, which may be moved vertically or rotated at will, with a nozzle which will throw a jet of water horizontally, is inserted in the tube and connected with a pressure system by a hose or other device. Manipulation of the jet bores a hole of sufficient size into which the pile may settle, the material thus removed passing upward and out with the water, by way of the inner tube. When this first section of the pile has reached a level in sinking that makes it necessary to lengthen it, courses are added to the shell, the tube is extended, concrete is placed, and sinking is resumed. The process of extension is repeated until the pile has reached the required depth, and then the space beneath is washed thoroughly and concrete is placed there as well as in the tube, making a column of uniform strength throughout its length, and in which the concrete can all have been properly handled and inspected.

Fig. 22 shows two piles being sunk. A man on the highest staging is manipulating the jet pipe with a wrench. The pile to the left has been sunk to a point where it requires to be extended. The inner tube has been lengthened, and courses are to be added to the outer shell. This will then be filled with concrete and the sinking will be resumed.

FIG. 22.—METHOD OF SINKING PILES, UNION MEAT COMPANY'S PLANT, NORTH PORTLAND, ORE.

FIG. 23.—METHOD OF FORCING A PILE BY LOADING.

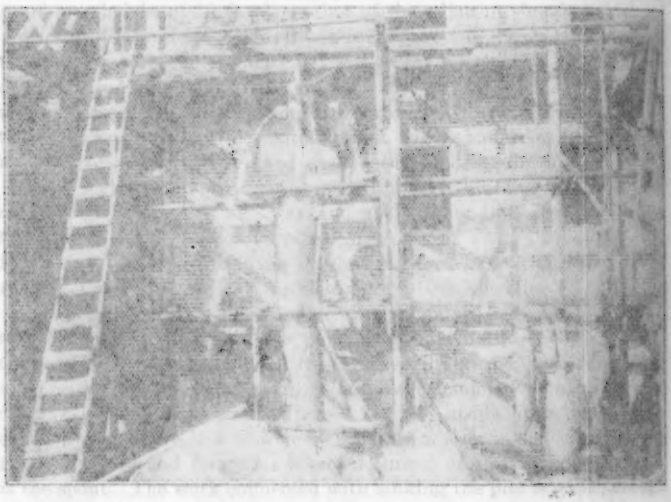


FIG. 24.—SAME PILE AS SHOWN IN FIG. 23, AFTER SINKING WAS COMPLETED.

FIG. 25.—CRACK IN WALL OF BUILDING CAUSED BY TENDENCY OF BLOCKING TO ROLL.

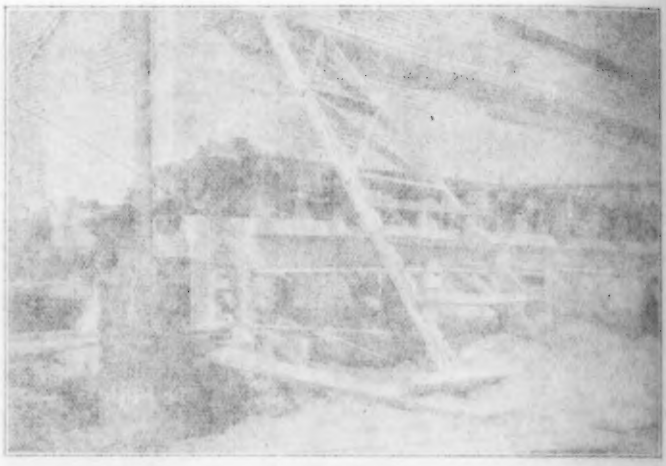


FIG. 22.—FRAME PILE AS SHOWN IN FIG. 23, AFTER DRIVING WAS COMPLETED.

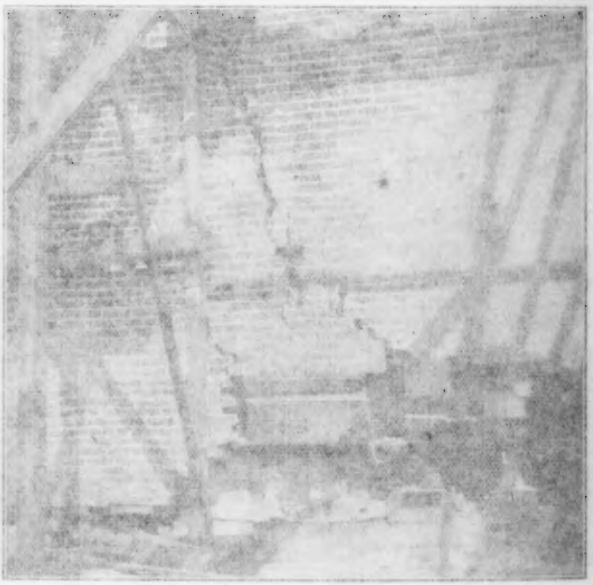


FIG. 23.—CRACK IN WALL OF BUILDING CAUSED BY TENDENCY OF BLOCKING TO HOLL.

Mr. Phillips.

Fig. 23 shows a pile which has reached a depth where skin friction offers too much resistance to be overcome by the weight of the pile alone, or even by a process which was called the "blow." This consisted in shutting off the water where it is seen escaping above and thus forcing it to find an outlet around the outside of the pile, thereby lessening the skin friction. When the "blow" was not effective, the pile, if on the outside of the building, was loaded as shown, and, if inside, was forced down with jack-screws.

Reinforcement may be used in such portions of the pile as may require it, but it was not necessary in the work described. In this work, 153 piles, ranging in diameter from 22½ to 30½ in. and in length from 108 to 112 ft., were sunk.

For the support of the walls of the main packing house, the piles were put down in pairs, each pair supporting a reinforced concrete beam which was to carry its share of the wall. When the piles required for the purpose were completed, and in advance of the completion of those intended to support the columns inside the building, the beams they were to support were poured, and in due time the weight of the walls was brought to a bearing on them.

Fig. 24 shows the same pile as Fig. 23, the sinking being completed. The lower end of this pile rests on cemented gravel, 116 ft. below the surface. The blocking and jack-screws show the method of holding up the brick walls while the foundation was sinking at the rate of 0.1 ft. per day. At the corner, inside the building, the mate of this pile has been sunk. A hole will be cut through the foundation, below the surface, and a reinforced concrete beam will be built on the two piles.

Fig. 25 shows one of several cracks in the walls of the building caused by the tendency of the blocking to roll. Such fractures had to be closed up after the building had been brought to a state of rest. After piers of brick set in cement mortar had been substituted for the wooden blocks and wedges, there was no further tendency to roll.

During the progress of the work this building had been sinking constantly, but at rates that varied according to the distance from one corner of the building on the end nearest the slough, which had remained stationary. When, finally, the walls were brought to rest on the new foundation, the subsidence at some points amounted to fully 2 ft.

The columns supporting the floors of the three departments of the main packing house were in three rows in each department. To carry these, two intermediate rows of piles were sunk in line with the original piers, and reinforced concrete beams resting on the intermediate piles carried the three columns in alignment with them.

The tank house, being a lighter structure, was handled in a different way. The concrete piles for it were set away from the old founda-

Mr. Phillips.

tion, and on these, continuous beams were poured. The building was moved over to the new foundation.

The work of erecting the buildings had progressed simultaneously with that of the new foundations. That it might be maintained in as nearly a level condition as possible, the walls and columns had been carried on jacks and oak wedges which were kept constantly adjusted. In spite of the care used to prevent it, the walls were badly cracked, and required considerable repairs.

There is no intention to convey the impression that there were no difficulties in sinking the piles, but such difficulties were readily overcome. It was seldom that a pile would follow the jetting as it proceeded. Instead, a cavity of some depth usually had to be washed out beneath, into which the pile would sink in due time. In general, the first 50 ft. or so of sinking was comparatively easy; but beyond that depth, in most cases, more or less urging had to be resorted to. If, however, larger piles had been necessary, or if a greater penetration had been required, or both, the work could still have been accomplished without undue difficulties.

Mr. Goodrich.

E. P. GOODRICH,* M. AM. SOC. C. E.—This paper appeals to the speaker more strongly with reference to incidental and exterior phenomena than with reference to the ingenious and eminently satisfactory method used in righting the elevator, which is the primary reason for the paper.

The first item of particular interest to the speaker is the demonstration that a structure of the size of this grain elevator, when built of reinforced concrete in accordance with the proper design, becomes a monolithic structure capable of being subjected to extraordinary stresses without seriously damaging the building. The settlement and righting of this elevator are almost identical with an experience concerning a grain elevator in Algiers, erected and righted by French engineers.

The moral of this story is an indirect demonstration of the need of caring for reverse moments in all bottoms, and at the points of intersection of bottoms and columns, together with the need for care in the design of columns to resist bending where need is shown. So long as reinforced concrete members were designed after the fashion of those made of timber or steel, difficulty was sure to arise, but, with the wider adoption of the cantilever, difficulties have disappeared and better structures been secured. The speaker has been interested in noting the widening use of the cantilever, even in steel and timber work, and believes that members thus designed are more efficient than almost any other device for the same purpose.

Reverting to the fact of the monolithic nature of large reinforced concrete structures, many instances may be cited of caissons designed

* New York City.

Mr. Goodrich.

for use as foundations for bridges, docks, etc., which have been handled, lifted on one side, tilted, and manipulated in almost any way found necessary without great difficulty.

The second point of interest refers to the earth pressure phenomena described. The wave that was thrust up, perhaps from some deep stratum, can be made the source of information with reference to earth pressure matters, which may prove of value through an analysis of its size, rate of formation, etc., if such data can be made available. For a long time the speaker has been advocating the making of careful explorations of sub-strata under large buildings, going down many feet below the customary foundation depth. Demonstrations of this need, in addition to that afforded by the Canadian Pacific grain elevator, were also mentioned by Robert B. Stanton, M. Am. Soc. C. E., in his discussion* on earth pressure formation in connection with a slide which took place on the Canadian Pacific Railroad within his observation.

The upheaval of the bottom of the Panama Canal is a gigantic demonstration of the same fact. On a much smaller scale, the upheaval of a steam shovel excavation in Cleveland, which occurred several years ago during the cutting through of a street, is pertinent.

It is the speaker's belief that properly made explorations and physical tests of the soil secured will make it possible to prophesy the possibilities of just such failures as occurred in connection with this grain elevator. Where such possibilities are indicated, obviously, engineers can take all needed precautions to prevent their occurrence. This belief has been applied by the speaker in several foundation investigations, and it is equally applicable to the analysis of soil conditions at Panama or Winnipeg. This paper is thus believed to be of wider interest than that covered in its main description of the mechanical methods used in combating the particular difficulty here encountered.

* *Transactions*, Am. Soc. C. E., Vol. LIII, p. 307.

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Paper No. 1364

INDUCED CURRENTS OF FLUIDS*

By F. zur Nedden, Esq.

With Discussion by Messrs. Clemens Herschel, Carl George de Laval, John C. Trautwine, Jr., and F. zur Nedden.

Synopsis.

The effects of internal and external friction on the movement of fluids can be traced in many instances to the induction of secondary currents which are co-existent with the main flow along geometric stream lines, and completely alter the resultant motion. The sources of the induced currents are the molecular forces at work in the marginal strata.

Induced Counter Currents.—These are generated when kinetic energy is transformed into pressure energy in diffusors.

The energy lost by the generation and maintenance of the counter currents is practically independent of the roughness of the wall.

Diffusors with geometrically similar cross-sections show a maximum of efficiency at a narrowly defined angle of dilatation. Actual figures are shown on Fig. 4. This angle increases slightly with increasing roughness and with increasing viscosity. It is no less harmful for the efficiency of the diffusor to have it opening out too quickly than to make it too slender. Spiral flow increases the efficiency of diffusors. The maximum of loss occurs, not at sudden enlargements, but in diffusors opening out at about 60 degrees.

* Presented at the meeting of October 20th, 1915.

Transverse Induced Currents.—These are generated where the direction of flow is altered, that is, in bends, elbows, etc. It is proved, both mathematically and by experiment, that their intensity depends in the first place on the ratio between radius of curvature and width of the vessel. (See Fig. 13.) This ratio is called degree of curvature. Two symmetrical transverse induced currents are generated in ordinary bends. The loss of head caused by their impact against each other—the restriction in free cross-section resulting from the transverse motion—and the loss of head due to repeated acceleration and retardation of every single vein of the current, are the three principal causes for the extra loss due to curvature. The roughness of the bend plays an unimportant part. The losses seem to be roughly proportional to the square of the speed. They are not confined to the bend proper. The readjustment to straight flow is usually connected with heavier loss than the adjustment to curved flow.

In Fig. 16 are tabulated the excess losses which must be expected to arise in American Standard long-turn elbows.

For speeds of flow above the critical limit, "equipotential" bends, shaped according to the classic conceptions of stream lines, cause a higher loss of head than simple circular bends of uniform rectangular cross-section.

Bends also cause a local decrease in static pressure (depression) which is often a multiple of their resistance. Fig. 18 gives an idea of the magnitude of these depressions as they occur in Standard elbows. Both losses and depression are more than doubled in **S**-pipes.

The following practical conclusions are derived from the foregoing facts:

Suction Pipes and Delivery Bends of Turbine Pumps.—The local depression caused by an elbow attached directly to the inlet of a low-lift pump can be detrimental to efficiency. At high rates of flow there is danger that gases may separate and interrupt continuity of flow if there are sharp turns in suction pipes.

When determining the efficiency of centrifugal pumps, care should be taken lest the gauge readings be influenced by the local depressions or excesses in pressure caused by bends. The use of decreasers for steadying the flow and removing local pressure differences is recommended.

Impellers.—The bearing which the induction of secondary currents may have on the transmission of energy in impellers cannot yet be defined.

The writer contends that it is possible to estimate the effects of the profile of the impeller on the distribution of pressure and velocity across the axial (longitudinal) section of the cells of the runner. The influence which the ratio of diameters must have on this distribution and on the efficiency and performance of the pumps is emphasized. As a means against the effects of excessive width of impellers the usefulness of radial guide-vanes at the entrance is questioned, and a suggestion is made for the introduction of an auxiliary partition of the cells and of a cylindrical guide in the entrance chamber. A modification of the usual application of velocity diagrams for determining the correct angle of the diffusor blades is recommended.

Diffusor Vanes.—Attention is drawn to Gibson's recommendations as regards proper curvature of diffusor vanes. The curvature of the axis of guide-channels is deemed advantageous. The dangers arising from a change in the sense of their curvature are emphasized.

Introductory.

Before entering into the subject, the writer wishes to state that this paper is directed more toward the practical consequences than toward an exhaustive treatment of the details connected with the induction of secondary hydraulic currents. Special reference is made to turbine pumps, but it is obvious that the results have just as important a bearing on the domains of water turbines, ship propellers, etc. As long as gases may reasonably be supposed to remain of constant density during their flow, the results are immediately applicable to fans, ventilators, turbo-blowers, etc.

There is a wide-spread misconception that, for all practical purposes, a sufficient insight into hydraulic phenomena may be gained by first applying the calculus to an ideal fluid free from internal or external friction and flowing along stream lines geometrically similar to the profile of the vessel, and then making good for the difference between the mathematical and the experimental results by introducing some simple coefficients covering the effect of friction. This mode of procedure would be perfectly admissible if the numerical quantities only of pressure and speed were affected by friction. This is not

the case. The effects of friction often cause the character of hydraulic phenomena to differ entirely from a flow along stream lines. Failure to realize this fact is mainly responsible for the frequent and gross discrepancies between practical and theoretical results of applied hydraulics. As an example, the writer would mention the conversion of kinetic energy into pressure by using diffusors, as compared with the reverse process in the nozzle.

Mathematics was able to throw considerable light on the reasons for these discrepancies. Investigations like those published by Navier, Stokes, and Boussinesq resulted in systems of fundamental equations which cover the influence of friction. Up to the present, however, the application of their equations to every-day practice has failed to crystallize into anything nearly as lucid as the application of the calculus to electrical phenomena. Even when based on the more complete equations of Navier, Stokes, and Boussinesq, hydraulic calculations do not reach the degree of approximation in predicting practical results of variations in design obtained by the laws of Ohm, Faraday, or Steinmetz.

In this plight designers are indebted to the hydraulic investigations, both mathematical and especially experimental, of the last 10 or 15 years for establishing a simplified and explicit new method of viewing and estimating the influence of certain typical changes in sectional area or direction of flow on the conversion of energy in hydraulic apparatus. The simplification consists in splitting up the complicated movements of the particles of non-ideal fluids into two components:

1.—A main movement along stream lines; and

2.—A secondary movement due to the influence of friction.

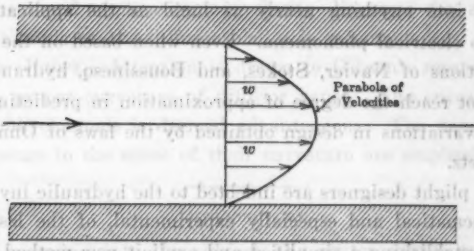
In other words, these investigations have established the fact that it is possible and admissible to look upon internal and external friction of non-ideal fluids as responsible for the induction of secondary currents, which, for convenience, will hereafter be referred to as induced currents.

The induced currents cause friction losses of their own, comparable to the phenomenon of self-induction of electric cables carrying alternating current. These additional and indirect losses could not be accounted for by a theory which considered one component only, namely, the main flow along geometrical stream lines.

The new development doubtless received its original impetus from Professor Osborne Reynolds'* discovery, in 1900, of turbulence in straight pipes. Professor Prandtl,† Chief of the Aero- and Hydrodynamical Laboratory at Goettingen, Germany, extended Reynolds' results to the flow in all kinds of pipes. Isaachsen,‡ who is manager and superintendent of a water turbine factory in Brunswick, Germany, was the first publicly to apply the new theory to the design of hydraulic machinery, especially water turbines.

I.—Induced Counter Currents.

1.—Critical Velocity.—All particles of a fluid flowing slowly through a straight pipe travel on straight lines (stream lines), according to a



DISTRIBUTION OF VELOCITY OF FLOW ACROSS THE CROSS-SECTION OF A PIPE AT A RATE OF FLOW BELOW THE CRITICAL LIMIT.

Fig. 1.

law which was experimentally found by Poiseuille§ and theoretically established by Hagenbach.‖ Those particles adjacent to the walls adhere to them, because the molecular forces of adhesion are stronger than the frictional (cohesive) forces between this, the marginal, and the next smaller concentric layer. The velocity is highest in the center and follows a parabolic law in decreasing to zero at the walls. (Fig. 1.) There is no relative movement between liquid particles and the wall. The resistance to flow is made up merely of the internal friction between the fluid particles (viscosity). The resistance is

* Papers on Mechanical and Physical Subjects, 2, Cambridge, 1900.

† Handwoerterbuch der Naturwissenschaften, Jena, 1913, *vide*: "Fluessigkeitsbewegung."

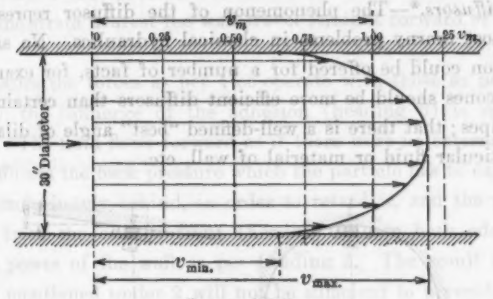
‡ *Zeitschrift* des Vereines Deutscher Ingenieure, Vol. 55, 1911, Nos. 6, 7, and 11.

§ Recherches expérimentales sur le mouvement des liquides dans les tubes des très-petits diamètres, Paris, 1844.

‖ Poggendorff's Annalen, 1860, I, p. 385.

independent of the roughness of the internal surface of the pipe. On this fact the working of viscometers is based.

When the rate of flow is increased beyond a certain "critical" speed, this ceases to hold true. The adhesive power of the wall is no longer strong enough to withstand the pull of the cohesive forces between the fluid particles. Consequently, the critical speed in every instance depends on the relation between the adhesive power of the wall (roughness and sectional circumference) and the viscosity of the fluid. Beyond the critical speed the strata immediately adjacent to the walls begin to move. The friction between them and the wall comes in as a



$v_{min.} = 0.57\ v_{max.}$
$v_m = 0.84\ v_{max.}$
$v_{max.} = 1.75\ v_{min.}$

DISTRIBUTION OF VELOCITY OF FLOW ACROSS THE CROSS-SECTION OF A PIPE ABOVE THE CRITICAL VELOCITY
(Mean Values of Results found by Williams, Hubbell, and Fenkell)

FIG. 2.

new source of losses. The velocity still is highest in the axis of the pipe, but the character of the flow has changed: the velocity follows an elliptical law in decreasing from the maximum in the center to a minimum at the wall.* (Fig. 2.) For water of 65 to 70° Fahr., this minimum is about 57% of the maximum.

The critical speed, for water of the same temperature and ordinary 4-in. iron pipe, is about 1 ft. per sec. In practically all technical instances the rate of flow is above the critical value.

2.—Nozzles.—It is well known† that all across the outlet of a smooth nozzle the velocity is so uniform, and so nearly $= \sqrt{2\,g\,H}$

* Williams, Hubbell and Fenkell, *Transactions*, Am. Soc. C. E., Vol. XLVII, p. 185.

† John R. Freeman, *Transactions*, Am. Soc. C. E., Vol. XXI, p. 365.

(where H is the combined velocity head and pressure, in feet, at the nozzle inlet) that the product of $\sqrt{2gH}$ and the area of the nozzle outlet can be taken to be a very accurate expression for the quantity of water passing through the nozzle per second. The explanation for this fact is that the particles deflected by the conical walls are continually getting into the way of those radially adjacent. By these impacts the latter convey as much energy (*vis viva*) to the former as is on the other side taken away from them by external friction. The influence of wall friction is thus automatically eliminated. The real flow comes remarkably near that of an ideal fluid.

*3.—Diffusors.**—The phenomenon of the diffusor represented one of the most thorny problems in classical hydraulics. No satisfactory explanation could be offered for a number of facts, for example: that straight cones should be more efficient diffusors than certain trumpet-shaped pipes; that there is a well-defined "best" angle of dilatation for each particular fluid or material of wall, etc.

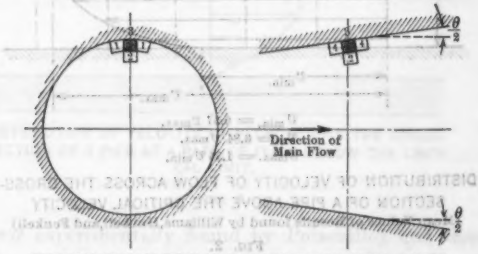


DIAGRAM OF MOLECULAR FORCES IN MARGINAL LAYER OF A DIFFUSOR

FIG. 3.

To get at the root of the problem, it is necessary to study the various molecular forces acting on a particle immediately adjacent to the wall. They can be described thus (Fig. 3):

1.—The forces of cohesion with the particles (1-1) tangentially adjacent on both sides. They pull in opposite directions and rectangular to the main flow (axis of diffusor). In straight, smooth pipes these forces are in equilibrium; their influence on the motion is zero.

* That is, increasers or pipes enlarging in cross-section: the reverse of a nozzle.

2.—The cohesion of the particle (2) radially adjacent and pulling in the direction of the main flow.

3.—The adhesion to the wall (3) pulling opposite to the direction of the main flow.

4.—The forces due to the inertia of the particles (4-4) immediately in front and immediately behind, when looking in the direction of the main flow.

When the fluid flows through a straight pipe at a constant rate, the result of the forces mentioned under 1 and 4 is zero, and the forces under 2 exceed (at speeds above the critical) those under 3. Consequently, the strata nearest the wall travel straight forward at a uniform speed.

In nozzles the forces as per 4 co-operate with those as per 2. Consequently, the influence of the adhesion (heading 3) is very nearly removed. The fluid flows forward as if there were no external friction.

In diffusors the back pressure which the particle has to exert against the one immediately behind, in order to retard it, and the pressure it receives from the one in front (heading 4), are both added to the adhesive power of the wall as per heading 3. The result is that the cohesion mentioned under 2 will not be sufficient to prevent the strata near the wall from reversing and flowing backward.

That this returning movement is not only limited to an infinitesimal layer near the walls, but that it permeates a considerable portion of the available cross-section may be made clear by an example chosen at random, say, a horizontal, conical increaser from 3 to 4 in. in diameter.

When 240 gal. per min. flow through such a diffusor, there will prevail a mean speed of about 11 ft. per sec. in the smaller and of about 6 ft. per sec. in the larger area. According to theory, the difference in pressure between the large and the small end should be $\frac{11^2 - 6^2}{2g} \cong 1.3$ ft. This formula neglects friction. In practice, the difference in a standard increaser will be about 1 ft., 0.3 ft. head being lost in the increaser.

The explanation for the excessive loss is found when taking into account, first, the influence of internal and external friction in the inlet pipe; and second, the same influences within the diffusor proper.

Turning to Fig. 2, it will be seen that the center vein of the stream flows at a rate of about 1.19 of the mean velocity, or at 13 ft. per sec.; the speed of those particles nearest the walls of the pipes will be about 0.57 of the maximum, or, in this instance, 7.4 ft. per sec. Even when not considering the friction within the diffusor itself, it is easy to show that a return current must necessarily be induced by this variation of the speed across the pipe section.

The velocity head of the center vein is: $\frac{13^2}{2\,g} \cong 2.6$ ft. As no energy is added during the flow through the diffusor, the velocity head at the outlet must be $2.6 - 1.0 = 1.6$ ft. (1 ft. being the increase in pressure observed), or the speed of this particular vein at the outlet will be $\sqrt{2\,g \times 1.6} \cong 10$ ft. per sec.

When singling out nearer the circumference another stream line entering with a speed of about 8.5 ft. per sec., we find that its velocity head is $\frac{8.5^2}{2\,g} = 1.11$ ft., which decreases by 1 ft., and therefore leaves only 0.11 ft. at the outlet, corresponding to a velocity of 2.7 ft. per sec. The area of this particular stream line would have to increase to about 3.15 times its original area $\left(\frac{8.5}{2.7}\right)$, whereas the rate of enlargement of the increaser is only about $\frac{4^2}{3^2}$, or 1.8. Particles entering with a velocity of about 8 ft. per sec. would arrive at the outlet with zero speed. The peripheral strata with speeds ranging from 8 ft. per sec. downward to 7.4 ft. will not be able to reach the outlet at all; they will, after exhausting their store of energy, reverse their direction and flow backward from the area of higher to that of lower pressure.

Considering now the effect of viscosity and adhesion in the diffusor proper, it is clear that in reality the individual veins of fluid cannot choose their way as independently as at first supposed. Returning particles will be reversed and re-accelerated by their friction against molecules moving at a greater speed. By this assistance they can overcome the friction with the pipe and the back pressure toward the inlet. The acceleration effected by the internal veins must be proportional to the area of contact between the respective strata, and proportional also to the viscosity.

The gradient of back pressure against which the fatigued particles have to be forced is a direct function of the angle of dilatation or

divergence, θ (Fig. 3). It is obvious that there will be one definite limiting value of θ for every fluid. Below this value, the external friction will exceed the viscous forces, and stop the counter currents. It follows, further, that this angle will be greater, the more viscous the fluid, and smaller, the smoother the inner surface of the pipe.

Under all circumstances, there will be a complicated exchange of energy between the different strata. This explains the excessive hydraulic loss connected with the transformation of kinetic energy into pressure.

The first to investigate more closely the extent (though not the exact nature) of these losses were Francis* and Fliegner.† With their results the engineering world rested content until at the beginning of this century the exigencies of turbine pump design pressed for a more intimate knowledge. Among the various investigations dealing with the subject those of Andres‡ and Gibson§ are the most noteworthy.

These scientists clearly realized the fact that the loss of head in divergent pipes is the result of two distinct components: the losses due to the primary or inductive and those due to the induced or secondary currents. Andres succeeded in separating these components. By combining the results of Andres with those of Gibson and several other independent investigators, the writer arrived at a graphical diagram, Fig. 4.

If the mean velocity $\left(v = \frac{9}{f}\right)$ of the water flowing through the throat of the diffusor be v_1 ft. per sec., and v_2 that prevailing in a certain enlarged cross-section, then, according to Bernoulli's theorem, there should be an increase in pressure between the two points of:

$$\Delta h = \frac{v_1^2 - v_2^2}{2g} \text{ (feet)} \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

If the increase is found in reality to be only $\Delta h'$, then the total loss of transformation $= V_d = \Delta h - \Delta h'$, and the "efficiency of the diffusor", as a percentage of Δh, can be expressed thus:

$$\eta_d = 100 \frac{\Delta h'}{\Delta h} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

* Lowell Hydraulic Experiments, New York, 1871, p. 219.

† *Zivilingenieur*, 1875, p. 97.

‡ "Versuche über die Umsetzung von Wassergeschwindigkeit in Druck", Forschungsarbeiten des Vereines Deutscher Ingenieure, Heft 76.

§ "Loss of Head Through the Flow of Water Through Divergent Pipes", *Proceedings*, Royal Society, London, Vol. 83, 1910; and *Engineering*, February 16th, 1912.

If V_d is expressed as a certain percentage of Δh, and, in this shape, is called V_d', then:

$$V_d' = 1 - \eta_d \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

It is possible to integrate for each particular diffusor, the loss in head,

$$h_r = \int d\, h_r \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

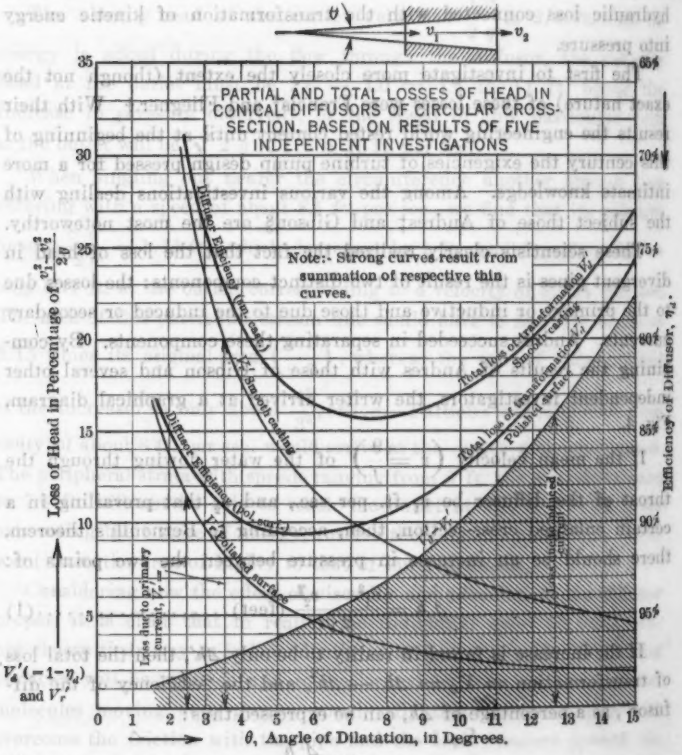


FIG. 4.

which would occur if each differential section of the increaser were part of a pipe of uniform cross-section, that is, if the peculiar additional loss resulting from the transformation of kinetic energy into pressure were non-existent. h_r, which may be called the loss due to the primary

current, can again be expressed as a percentage, V_r' of the theoretical increase in pressure:

$$V_r' = \frac{100\, h_r}{\Delta h} \qquad (5)$$

$V_r' =$ infinity where $\Delta h = 0$, that is, in straight pipes of uniform cross-section with angle of dilatation $\theta = 0$. $V_r' = 0$ when $h_r = 0$, that is, when the length of the diffusor $= 0$, $\theta = 180$ degrees. This is the case of sudden enlargement.

Between these two extremes the value of V_r' depends on roughness, width (hydraulic radius), etc., in true proportion with the ordinary coefficients of friction in pipes. V_r' is represented in Fig. 4 by the faint hyperbolic curves.

When subtracting the wall friction loss, V_r', from the values experimentally established for the total loss of head, V_d', we find the remainder, $V_d' - V_r'$, or "enlargement loss", due to the loss caused by the secondary or induced currents.

If the new theory is right, then this enlargement loss must be independent of the roughness of the walls, and merely a function of the viscosity and the angle of dilatation, θ. As the several hundreds of experiments on which Fig. 4 is based were all carried out with water at temperatures of about 60 to 70° Fahr., that is, of equal viscosity, subtraction of the wall friction losses from the total losses must lead to practically the same values, whether the diffusors were rough or smooth. This tallies with a remarkable degree of exactitude: This is shown by the curve for $V_d' - V_r'$, marked by hatching in Fig. 4.

Fig. 4 is of considerable interest to the designer of diffusors. It emphasizes most forcibly two important facts:

1.—The degree of polish given to the walls of a diffusor is not of paramount importance—provided only that the angle of dilatation is right: the best diffusor efficiency obtainable by an unmachined though smoothly cast taper pipe is about 84%, and a high polish, under the best conditions, can only add another 6% in the efficiency of the diffusor. On the other hand, a polished diffusor diverging at an angle of 14° is less efficient than an unmachined increaser of 8° divergence.

2.—It is just as harmful to the diffusor efficiency to have a diffusor opening out too quickly as to make it too slender. Though, for example, the best results are obtained with an unmachined taper pipe diverging at 8°, 10% of diffusor efficiency is lost by designing

the angle, θ, either 3 or 14½ degrees. Generally, the designer will be much more tempted to choose a large angle of dilatation than a small one, as the latter implies a great axial length of the apparatus.

It is interesting to note, and quite consistent with the theory of induced hydraulic currents, that the diffusor losses, V_d', reach a maximum of more than 100%, not at 180° of dilatation, but at about 60°, and then fall slowly to about 100% at 180 degrees.* At angles larger than 60° obviously the counter currents find an area large enough to develop freely and without materially disturbing the main flow. As the difference, $\frac{v_1^2 - v_2^2}{2g}$ (see Equation (1)), is, in these instances, entirely used up for overcoming the local counter currents, there remains no back pressure which would reverse the flow near the farther end of the diffusor.

It is also consistent with the new theory that the experiments prove rectangular cross-sections to cause a greater loss in head than circular ones. Evidently, in the corners of the rectangles, counter currents can develop more freely.

Perhaps the strongest proof for the thesis that the induced currents are to blame for the inferior efficiency of diffusors is to be found in the fact that the efficiency of one and the same diffusor improves when these currents are artificially hampered, and that it reaches almost 100% when the diffusor is forcibly swept clean from any secondary currents. This can be done by imparting to the fluid a rotary motion around the axis of the diffusor. The centrifugal forces thus developed overcome the molecular resistances of wall friction and inertia, and create a resultant nearly parallel to the wall. The phenomenon then closely resembles in effect the flow through a nozzle.

This proof was elegantly furnished by Andres. The water before entering his experimental diffusors had to pass either:

a.—Twenty strainers of fine-meshed wire gauze, securing an absolutely parallel flow of the individual stream lines; or,

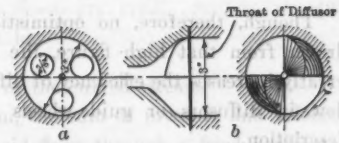
b.—One such strainer; or,

c.—Nothing; or,

d.—A blind flange, 3 in. in diameter, with three equidistant round holes, each ⅞ in. in diameter—(Fig. 5*a*) an arrangement called an "eddying sheet"; or,

* *Engineering*, February 16th, 1912, Fig. 1, p. 205.

e.—A "propeller sheet", that is, a piece of sheet iron cut along two rectangular diameters and then having two opposing quadrants bent open, thus giving the stream lines a twist (Fig. 5*b*).



"EDDY SHEET" (*a*) AND "PROPELLER SHEET" (*b*), AS USED BY ANDRES

FIG. 5.

Andres read the pressures in the throat, that is, after the liquid had passed the respective strainers or sheets, and at the end of the diffusor. Table 1 contains his results.

TABLE 1.—PERCENTAGE OF EFFICIENCY OF TRANSFORMATION OF KINETIC INTO PRESSURE ENERGY.

Diffusor.	Polished Castings.					Smooth castings unmachined.
	20 strainers.	1 strainer.	Nothing (normal).	"Eddy sheet."	"Propeller sheet."	
Circular cross-section. $\theta = 8°20'$.....	86.5	87.1	88.3	92.5	98.9	85.4
Rectangular (pyramidal) diffusor.....	67.0	66.5	71.9	73.2	70.6	....

The increase of efficiency effected by introducing a spiral flow is very striking. With reference to the high figure, 98.9%, obtained by using the "propeller sheet" in connection with the conical diffusor, the experimenter says:

"Though this figure is based on readings made with every conceivable care (as are all the others), doubts must be expressed as to its correctness. The loss due to the primary current alone must be about 4 per cent. [See Fig. 4.]

"The apparent inconsistency is removed by considering that the calculation of η_d is based on a value of the velocity, v_1, in the throat, which itself is found by dividing the quantity per second through the cross-section of the throat. This would involve that the stream lines be parallel to the axis, which they, of course, are not. The actual mean velocity is bound to be higher and therefore the value of

$$\Delta h = \frac{v_1^2 - v_2^2}{2g}$$

greater.

"However, it is impossible even to guess at the approximate value of the tangential components which should be geometrically added to the calculated mean velocity."

Though, therefore, no optimistic practical conclusions should be drawn from that high figure, the fact remains that the spiral flow greatly increases the efficiency of diffusors. As will be shown later, the flow in diffusors or guide-wheels of centrifugal pumps is of this description.

II.—Transverse Induced Currents.

Transverse induced currents, namely, currents circulating in a direction rectangular to the axis of the primary or inductive current, occur wherever the direction of flow is altered. The most notable example is the bend or elbow. The channels between two successive blades in the runner of a turbine or a turbo-pump or turbo-compressor represent bends curved in two planes. The induced currents resulting from this complicated curvature have an important bearing on the design of these machines.

Before entering into the discussion of experimental results, it is essential to make a short mathematical survey of the ground to be covered.

1.—The "Law of the Rotating Water".—If an ideal fluid flows through a circular bend of rectangular cross-section, then, according to Isaachsen's investigations,* the velocities of the stream lines are inversely proportional to their radii of curvature. If r_a and r_i represent the outer and the inner radius, respectively, and w_a and w_i the corresponding velocities at these radii (Fig. 6), the following equation holds good:

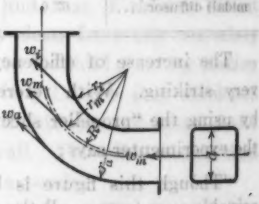


Fig. 6.

$$\frac{w_a}{w_i} = \frac{r_i}{r_a} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

or,

$$w_a\, r_a = w_i\, r_i = w_m\, r_m = C \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

The constant, C, is found by determining the radius, r_m, where, according to this result, a velocity, w_m, must occur, which is equal to the mean velocity $\left(w = \frac{9}{f}\right)$ in a straight channel of identical cross-

* *Zivilingenieur*, 1894-96.

section. This radius was called by Pfarr* the "neutral radius", and works out of the equation:

$$r_m = \frac{d}{\text{nat. log.} \frac{r_a}{r_i}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

where $d = r_a - r_i$ = radial width of the curved stream.

According to Bernoulli's theorem, the sum of pressure and velocity head remains constant when the fluid flows through a fixed horizontal channel.

The pressure must be lowest where the radius is smallest, and it is particularly noteworthy that, at the smallest radius, the pressure must be expected to be lower than at the neutral radius, that is, lower than in the straight ducts leading to and fro. The difference of the inside and outside pressure should be in feet:

$$\Delta p = \frac{w_i^2 - w_a^2}{2g} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

w_i and w_a follow from Equations (7) and (8).

The minimum pressure in the bend should then be lower than the pressure in the adjacent straight portions of the channel by:

$$\Delta p' = \frac{w_i^2 - w_m^2}{2g} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

where again w_i and w_m follow from Equations (7) and (8).

It should be emphasized again that this "law of the rotating water" was originally developed for ideal fluids. Without modification, it must be applied only to conditions of flow which agree in character with those of an ideal fluid, for example, flow below the critical velocity. Obviously, it is possible to apply it to a marginal layer, all particles of which flow forward in the same direction with a uniform speed. As this application furnishes the basis for the explanation of the origin and intensity of transverse induced currents, the writer felt encouraged to follow up the consequences of Equations (6) and (9) a little farther. He found a new law which is remarkably well substantiated by the experimental facts:

When calling "degree of curvature" the ratio, $\frac{R}{d}$ (where R is the radius of the geometrical axis, and d is the radial width (diameter) of the bend), this new law can be expressed thus:

* "Die Turbinen", 1st ed., p. 77; 2d ed., p. 86.

The total pressure difference, Δp, and the maximum depression, $\Delta p'$, are constant for a constant degree of curvature and constant mean velocity of flow. They are direct functions neither of the width of the bend nor of the radius of curvature.

Proof.—If $\frac{R}{d} = \text{const.}$, then

$$d = \frac{1}{\text{const.}} \times R = k \times R.$$

k is the reciprocal of the degree of curvature.

Now

$$r_i = R - \frac{k \times R}{2} \quad \ldots\ldots(11a)$$

and

$$r_a = R + \frac{k \times R}{2} \quad \ldots\ldots(11b)$$

Equation (7) leads to:

$$w_i = \frac{w_m \times r_m}{r_i} \quad \ldots\ldots(12a)$$

$$w_a = \frac{w_m \times r_m}{r_a} \quad \ldots\ldots(12b)$$

With these expressions, Equation (9) assumes the shape:

$$\Delta p = \frac{w_i^2 - w_a^2}{2g} = \left[\left(\frac{w_m r_m}{r_i}\right)^2 - \left(\frac{w_m r_m}{r_a}\right)^2\right] \frac{1}{2g}$$

$$= \frac{w_m^2}{2g} \times r_m^2 \left(\frac{1}{r_i^2} - \frac{1}{r_a^2}\right) \quad \ldots\ldots(13)$$

By introducing $k = \frac{d}{R}$ and the value for r_m as per Equation (8), a rather formidable looking equation is derived:

$$\Delta p = \frac{w_m^2}{2g} \times \left[\frac{\dfrac{R\,k}{R + \dfrac{k\,R}{2}}}{\text{nat. log.} \dfrac{R + \dfrac{k\,R}{2}}{R - \dfrac{k\,R}{2}}}\right]^2 \times \left[\left(\frac{1}{R - \dfrac{k\,R}{2}}\right)^2 - \left(\frac{1}{R + \dfrac{k\,R}{2}}\right)^2\right] = \frac{w_m^2}{2g} \times R^2 \left(\frac{k}{\text{nat. log.} \dfrac{1 + \dfrac{k}{2}}{1 - \dfrac{k}{2}}}\right)^2$$

$$\times \frac{1}{R^2} \left[\left(\frac{1}{1 - \dfrac{k}{2}}\right)^2 - \left(\frac{1}{1 + \dfrac{k}{2}}\right)^2\right] \quad \ldots\ldots(14)$$

In this equation, R^2 cancels.

Therefore:

$$\Delta p = \frac{w_m^2}{2g}\left(\frac{k}{\text{nat. log.}\dfrac{1+\frac{k}{2}}{1-\frac{k}{2}}}\right)^2 \times \left(\frac{1}{\left(1-\frac{k}{2}\right)^2} - \frac{1}{\left(1+\frac{k}{2}\right)^2}\right) \quad \ldots (15)$$

In the same way, an expression is found for the maximum depression:

$$\Delta p' = \frac{w_m^2}{2g}\left[\left(\frac{k}{1-\frac{k}{2}} \times \frac{1}{\text{nat. log.}\dfrac{1+\frac{k}{2}}{1-\frac{k}{2}}}\right)^2 - 1\right] \quad \ldots\ldots (16)$$

Both Δp and $\Delta p'$ are, therefore, quite independent of R or d.

Only the degree of curvature, $\frac{R}{d}$, or its reciprocal, k, and the mean velocity of flow determine the range of secondary pressures formed in the marginal layers of a bend.

These formulas may be applied only as long as the fluid may be supposed actually to follow the inner curvature of the walls; that is, only as long as the bend is not too sharp. They do not, as a rule, apply to short-turn elbows or tees. Professor Pfarr's experiments, published by Dr. Lell,* prove the law to be still applicable when $k = 1$. $k \leqq 1.2$ should represent the upper limit.

2.—Induced Currents in Bends (Long-Turn Elbows).—We now enter into a general survey of the conditions which develop when a non-ideal fluid enters a bend. Fig. 2 shows that in a straight pipe the central vein flows about 75% faster than the particles at the wall. When entering a bend, the molecules in the center will develop the strongest centrifugal force $\left(\frac{m w^2}{R}\right.$ being proportional to the square of $\left.w\right)$, or, in other words, will have a greater inertia to keep them flowing in the same direction and not be deviated before meeting strong resistance. They will dislodge the particles in their way, which, in turn, must find a way to flow toward the center of curvature. By thus following a curved path of smaller radius, they, in turn, gain in centrifugal force, while the dislodging particles travel on curved paths of larger radius and gradually lose energy. A moment later, therefore, dis-

* This is shown later.

lodgers and dislodged ones change their rôles and swing back. These oscillations are very distinctly visible in Fig. 12.*

The path of any single particle is approximately spiral, that is, it is the result of the co-operation of a main forward flow and a rectangular component: the circular or transverse induced current.

The pressure, of course, varies in accordance, being highest where the retardation of particles is strongest, and *vice versa*. Fig. 7 is a diagram based on experimental results as found by Dr. von Cordier.†

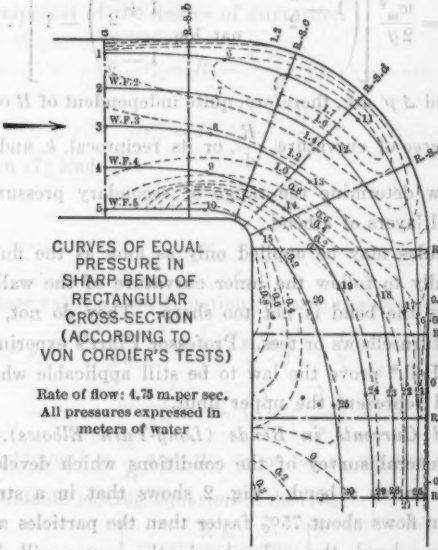


FIG. 7.

It shows clearly an elevation of pressure of 1.7 m. (about 5½ ft.) on the outer wall at Radial Section *d* (*R. S. d*) and the corresponding depression of minus 0.8 m. (or 2½ ft.) on the inner side

* Gardner S. Williams, M. Am. Soc. C. E., has kindly drawn the writer's attention to a very distinct exhibition of these oscillations in the tests made by Messrs. Williams, Hubbell, and Fenkell, at Detroit in 1901 and 1902. Referring to *Transactions*, Am. Soc. C. E., Vol. XLVII, April, 1902, p. 152, there will be noted in Fig. 77 a plotting of gauge readings taken at two extremities of the horizontal diameter of a 16-in. pipe, which is indicated on p. 149 of that paper. These observations were taken 213 ft. down stream from the termination of the first 90° bend at Cavalry Avenue and Porter Street, which had a radius of 5 ft. It will be noted that the maximum pressure varied from side to side as the velocities changed.

† "Strömungsuntersuchungen an einem Rohrkrümmer," ed. by Oldenbourg, Munich, 1910.

between Radial Sections f and g. The water arriving from "up hill" then by its impetus increases the pressure at the inner side. At *R. S.* i, the pressure has already increased again to 0.3 m., or 1 ft. above zero. The depletion of the outer strata of the bend, meanwhile, has caused the pressure there to fall from 5½ ft. above to about 4 in. (— 0.1 m.) below zero. (These fluctuations continue far into the straight duct following the bend. The process and the losses resulting therefrom are not at all confined to the bend proper. This fact makes the determination of what should be termed "loss due to curvature" a very difficult matter.)

It should be emphasized that the experimental bend used by von Cordier really was a short-turn elbow, with the coefficient, $k \cong 1.4$. The pressure difference created by it, therefore, cannot be expected to tally with the result of Equation (15).

This and other reasons caused Professor Pfarr, of water turbine fame, to start a thorough investigation of the flow through bends. A very extensive series of tests was carried out under his supervision, and after Pfarr's recent death, published* by Dr. Lell of the Darmstadt Technical University, Germany. These tests will rank forever among the classical feats of experimental engineering. It is impossible to go into any details of these remarkable investigations, as they fill a voluminous report. An impression of the thoroughness of the work performed may be gained from Fig. 13 of Lell's paper.† Water flowed from a tank, through a perfectly level and smooth rectangular duct, into the experimental bend ($k = 1.0$) which could be covered with interchangeable plates of various materials. The pressures were measured by several hundred stand-pipes, and connected by pipes of special design to rows of small borings at the inner and outer curved wall and on both covers of the bend. The walls of the straight ducts leading to and away from the experimental bend were also equipped with rows of test borings. Further, by introducing Pitot tubes of special design, the pressures within any stratum could be measured at any desired point with a very high degree of exactitude. The most elaborate precautions were taken against any errors, the readings being checked and

* "Beitrag zur Kenntnis der Sekundärströmungen in gekrümmten Kanälen," *Zeitschrift f. d. ges. Turbinen Wesen*, 1914, Nos. 9, 19, 20, 21.

† It is regretted that it has been impossible to secure, for reproduction in this paper, copies of the photographs used in illustrating Dr. Lell's paper.

rechecked. The difference of friction caused by a surface having hundreds of small openings as compared with a smooth surface also was carefully tested. It was found, in the arrangement tested, to be negligible. It is needless to state that all refinements of up-to-date scientific investigation were scrupulously applied.

In addition to the measurements, experiments were carried out with a glass cover-plate.

Air was introduced through special borings under a slight pressure. This method furnished an excellent means of studying the currents in detail. Photographs also were taken of these air-streams, and clearly show the deviation of the stream lines from the concentric shape assumed by the theory of stream lines.

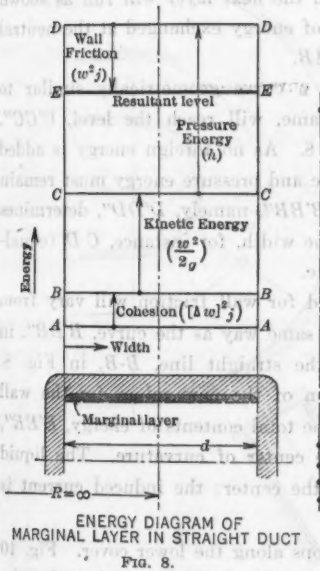
These experiments revealed the formation of two symmetric induced currents. As this result represents the foremost point gained in the knowledge of induced currents of fluids, it should be described here a little more in detail. The foregoing reasoning with regard to the struggle between dislodging and dislodged particles can only serve to convey a general idea of the nature of the phenomenon. It really leaves unanswered the crucial questions: Do the particles simply whirl around in an irregular manner? Is the way by which they travel from one side of the pipe across to the other dependent merely on accidental influences which we may never hope to disentangle? Or, do the dislodged particles travel along certain well-defined, typical paths, thus replenishing the inner strata in a methodical manner and forming a stream which may justly be termed an induced current? And, if so, where does that current start, and why must it be reproduced in every case?

In answering these questions the writer wishes to emphasize that the following considerations were not the base but the outcome of the experiments.

It was found that the induced currents originate in the strata nearest the covers and gradually increase in volume and intensity while the main current flows forward. When considering a very thin marginal layer along the upper or lower cover of the bend, the friction against the outer or inner curved wall may reasonably be disregarded. At the entrance to the bend all particles of this layer travel forward in about the same direction, and with reasonably the same

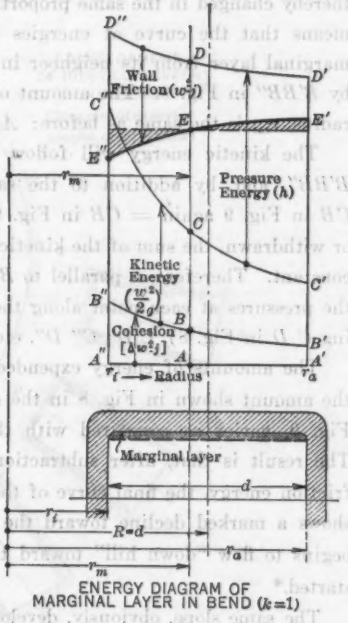
speed. The conditions on which the law of the rotating water is based exist with great approximation for this layer.

Referring to the foregoing survey for the various molecular forces pulling or pushing at each particle, the writer developed an energy diagram (Fig. 8), for the marginal layer under consideration. As long as the fluid flows through a straight duct or pipe (Fig. 8), each particle of the marginal layer receives, through the adhesive forces of



ENERGY DIAGRAM OF MARGINAL LAYER IN STRAIGHT DUCT

FIG. 8.



ENERGY DIAGRAM OF MARGINAL LAYER IN BEND ($k=1$)

FIG. 9.

the next layer, an amount of energy corresponding to, say, A-B; for the particles of that new layer also all flow forward with a uniform speed which, however, is slightly higher. Each particle of the marginal layer represents, by virtue of its velocity, a velocity head or kinetic energy of $\frac{w^2}{2g} = BC$, and, by virtue of its pressure, the potential energy, CD. It spends, in the form of wall friction, an amount of energy, DE.

As all particles of each of the two layers travel with uniform speed, the energy lines, *B-B*, *C-C*, *D-D*, and particularly the resultant, *E-E*, are level: The contents of the layer under consideration has no tendency to flow either to the right or to the left, but straight on.

As soon as the layer enters the curved part of the pipe, the velocities change in accordance with the law of the rotating water. The absolute differences in speed along the width of two adjacent liquid layers are thereby changed in the same proportion as the speeds themselves. This means that the curve of energies conveyed to each particle of the marginal layer from its neighbor in the next layer will run as shown by *B'BB''* in Fig. 9. The amount of energy exchanged at the neutral radius, r_m, is the same as before: *AB*.

The kinetic energy will follow a curve geometrically similar to *B'BB''*, and, by addition to the same, will reach the level, *C'CC''*. *CB* in Fig. 9 again = *CB* in Fig. 8. As no foreign energy is added or withdrawn, the sum of the kinetic and pressure energy must remain constant. Therefore, a parallel to *B'BB''*, namely, *D'DD''*, determines the pressures at each point along the width, for instance, *C D* (equaling *C D* in Fig. 8), *C'D'*, *C'' D''*, etc.

The amounts of energy expended for wall friction will vary from the amount shown in Fig. 8 in the same way as the curve, *B'BB''*, in Fig. 9, varies as compared with the straight line, *B-B*, in Fig. 8. The result is that, after subtraction of the new values for the wall friction energy, the final curve of the total contents of energy, *E'EE''*, shows a marked decline toward the center of curvature. The liquid begins to flow "down hill" toward the center: the induced current is started.*

The same slope, obviously, develops along the lower cover. Fig. 10 is a descriptive diagram reproducing the indications given by the streamlets of air bubbles pressed into Pfarr's experimental bend. The rectangles shown are the cross-sections of the bend along the indicated radii, and turned by 90° into the drawing plane. Immediately

* Strictly speaking, the assumption of separate layers of flow is not absolutely in harmony with the conception of viscosity, as defined by Newton, nor with the characteristics of a turbulent movement, as established by Professor Reynolds. Still, this simplification is highly serviceable, and it is admissible as long as it does not contradict experimental evidence, but only serves to illuminate it. It is to be borne in mind, however, that no conclusions must be drawn from the assumption of flow in layers, unless they are corroborated by experiment.

after the fluid has entered the bend the situation is shown by the diagram marked 1. The two layers next to the covers show a distinct component of flow, as indicated by the arrows. A moment later, the particles contained in the original layer, by their momentum and owing to the general outward tendency of the flow of the contents, travel downward and upward along the inner vertical wall, as shown by 2.

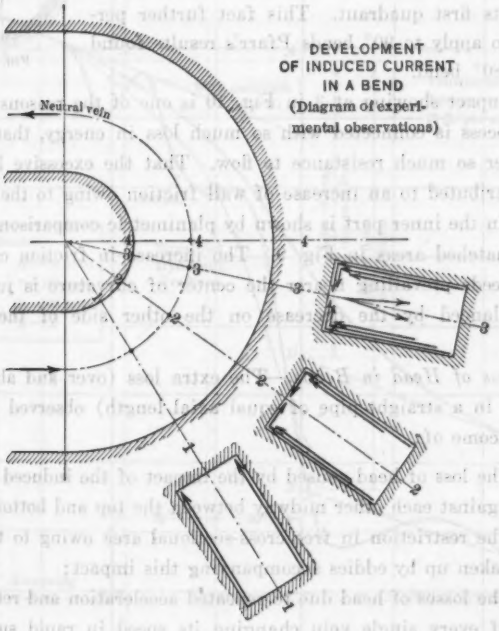


FIG. 10.

New layers come into contact with the upper and lower cover-plates and follow the paths taken by the first layers. At the same time, by the radial movement of all these particles, a radial pull is exerted on the next inner strata indicated in the diagram at 2 by the shorter horizontal arrows. Soon the state of affairs suggested at 3 is reached. The streams meet with considerable impetus half way between the upper and lower cover.

It is easy to show that, in a bend of circular cross-section, this impact will not be less hard: Fig. 11.*

After a 90° curvature has been traveled, the induced current is well under way. Nothing new in principle happens, nor do the radial components materially increase in strength. This explains the fact, already observed by Weisbach, that the extra resistance caused by a circular pipe or coil does not exceed that of its first quadrant. This fact further permits us to apply to 90° bends Pfarr's results found with a 180° bend.

FIG. 11.

The impact showing at 3 in Fig. 10 is one of the reasons why the whole process is connected with so much loss in energy, that is, why bends offer so much resistance to flow. That the excessive loss must not be attributed to an increase of wall friction owing to the increase in speed in the inner part is shown by planimetric comparison between the two hatched areas in Fig. 9. The increase in friction caused by higher speeds prevailing nearer the center of curvature is just about counterbalanced by the decrease on the other side of the neutral vein (*E*).

3.—Loss of Head in Bends.—The extra loss (over and above that occurring in a straight pipe of equal axial length) observed in bends is the outcome of:

1.—The loss of head caused by the impact of the induced currents against each other midway between the top and bottom;

2.—The restriction in free cross-sectional area owing to the space taken up by eddies accompanying this impact;

3.—The losses of head due to repeated acceleration and retardation of every single vein changing its speed in rapid succession.

The preponderance of these losses which are due to imperfect exchange of molecular energies, indicates that the roughness of the walls can only have a very subordinate influence on the resistance due

* Indeed, the impact should be expected to be even harder in a circular cross-section. In a rectangular cross-section the vertical strata along the inner wall are passive, and form a baffle for particles which are pressed down or up by the forces developing in the upper or lower marginal layer. In a circular cross-section, however, fresh impetus is added to the induced currents on their whole way until they reach the point of smallest radius, which coincides with the point where the two currents clash against each other.

to curvature. This important fact is substantially shown by experiments to which the writer will presently refer in detail.

The last source of losses is the most prominent of the three and is most vividly accentuated in the diagram, Fig. 12.

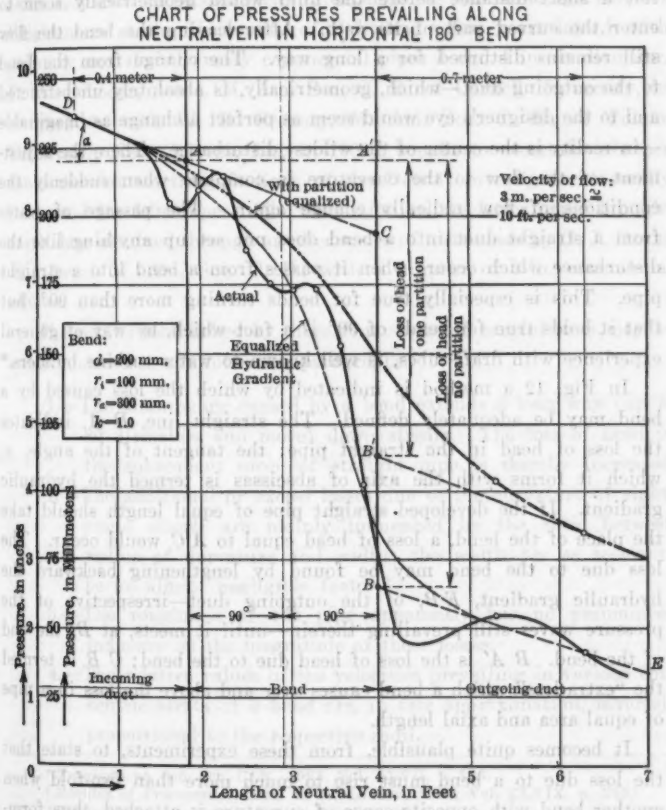


FIG. 12.

Dr. Lell measured the pressures prevailing all along the calculated "neutral vein" of Pfarr's experimental bend, and plotted them as ordinates over the development of the neutral vein as axis of abscissas. Fig. 12 represents his results for one speed, namely, 10 ft. per sec. The plottings for 20 and 30 ft. per sec. are very nearly identical with

this one, if their scales are reduced to $\frac{1}{4}$ and $\frac{1}{9}$, respectively. It follows that the total loss of head due to a bend should be expected to be pretty nearly in proportion to the square of the speed of flow.

Fig. 12 shows that the influence of the bend is already making itself felt a short distance before the fluid would geometrically seem to enter the curved part of its path. After leaving the bend the flow still remains disturbed for a long way. The change from the bend to the outgoing duct—which, geometrically, is absolutely unobstructed and to the designer's eye would seem as perfect a change as imaginable—in reality is the center of the wildest disturbance. There the adjustment of the flow to the curvature is complete when suddenly the conditions of flow radically change again. The passage of water from a straight duct into a bend does not set up anything like the disturbance which occurs when it passes from a bend into a straight pipe. This is especially true for bends turning more than 90°, but that it holds true for bends of 90° is a fact which, by way of general experience with draft-tubes, is well known to water turbine builders.*

In Fig. 12 a method is indicated by which the loss caused by a bend may be adequately defined. The straight line, $D\ A$, indicates the loss of head in the straight pipe; the tangent of the angle, α, which it forms with the axis of abscissas is termed the hydraulic gradient. If the developed straight pipe of equal length should take the place of the bend, a loss of head equal to $A'C$ would occur. The loss due to the bend may be found by lengthening backward the hydraulic gradient, $E\,B$, of the outgoing duct—irrespective of the pressure waves still prevailing therein—until it meets, at B, the end of the bend. $B\ A'$ is the loss of head due to the bend; $C\ B$ is termed the "extra-loss" which a bend causes over and above the loss in a pipe of equal area and axial length.

It becomes quite plausible, from these experiments, to state that the loss due to a bend must rise to much more than two-fold when another bend with opposite sense of curvature is attached, thus forming an **S**. The same will hold true as regards the lowest local depression occurring in an **S**.

a.—Loss in Bends of Circular Cross-Section.—It is interesting to compare the results thus far obtained with the conclusions arrived

* See also the experiments by Messrs. Williams, Hubbell, and Fenkell.

at by a number of investigators, both in the United States and abroad. Messrs. Williams, Hubbell, and Fenkell, at the beginning of this century, carried out a series of very careful tests with large bends (12 to 30 in. in diameter) forming part of the water supply system of Detroit, Mich.* A second series of experiments was made by Professor Brightmore, at the University of Calcutta, a few years later,† with ordinary and slightly rusty commercial pipes 3 and 4 in. in diameter. E. W. Schoder, Assoc. M. Am. Soc. C. E., of Cornell University, in co-operation with W. B. Gregory, M. Am. Soc. C. E., of Tulane University, New Orleans, La., in 1909,‡ sought to ascertain whether the somewhat startling findings of the Detroit investigators actually held good for 6-in. pipes. The discussion on the latter paper brought forth data obtained through another series of tests made by G. J. Davis, Jr., Assoc. M. Am. Soc. C. E., with a number of 2-in. commercial elbows.

The results established by these four series of careful tests may be compounded into the following paragraphs:

1.—The disturbance caused by a bend extends a long way (up to 80 diameters and more) down stream. The loss of head in the subsequent piece of straight pipe is thereby increased.

2.—The additional or excess losses due to the curvature of right-angle elbows are mainly influenced by the ratio between radius of curvature and width; the width *per se* seems to be an almost negligible factor.§

3.—The roughness of the pipe apparently has no pronounced influence on the magnitude of these losses.

4.—The relative values of the velocities prevailing in various concentric strata of a bend are, in fair approximation, inversely proportional to the respective radii.

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* *Transactions*, Am. Soc. C. E., 1902, Vol. XLVII, p. 183.

† *Minutes of Proceedings*, Inst. C. E., London, 1907, Vol. CLXIX, p. 315.

‡ *Transactions*, Am. Soc. C. E., Vol. LXII, p. 67.

§ It may not be out of place to refer to Weisbach's classical formula for the resistance to flow by a bend, namely:

$$H_f = \left[0.13 + 0.16 \left(\frac{d}{R}\right)^{3.5}\right] \frac{w^2}{2g}.$$

Though, by the foregoing tests, the coefficients of this formula are shown to be inaccurate, its structure indicates that Weisbach was well aware of the paramount importance of the ratio, $\frac{d}{R}$.

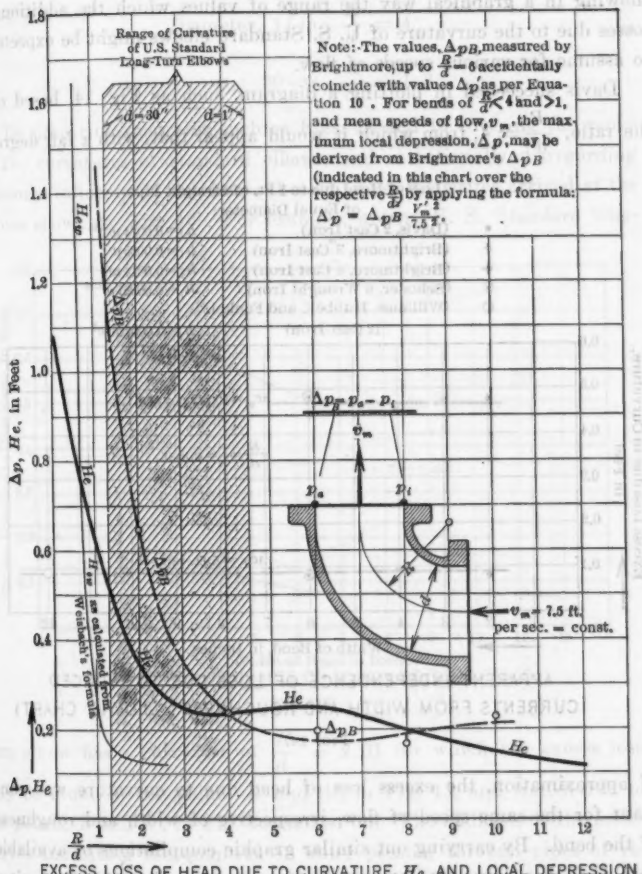
Neither of these investigators apparently knew of Isaachsen's and Pfarr's theories, and the latter's experiments were made at a later date than theirs. The coincidence of the results, therefore, is very convincing proof of their correctness.

The American and English investigations amplify the German results in one important direction. They furnish an experimental basis for the variation of the loss in function of $\frac{R}{d}$. Fig. 13, which is mainly plotted from Brightmore's results, reveals the fact that this function is rather complicated. When the Detroit experimenters first established a branch of the H_e-curve, Fig. 13, they viewed the results with considerable doubt. They themselves considered it rather queer that the losses should have a pronounced minimum for $\frac{R}{d} \sim 3$ and a secondary maximum at $\frac{R}{d} \sim 6$.

Their findings, however, were corroborated by all subsequent investigators, Brightmore arriving at his results apparently without knowing of theirs.

When comparing this curve with the results obtained from diffusors at various angles of dilatation, the similarity in character is most suggestive. The explanation should lie in the fact that in both cases the additional losses, being mainly due to induced currents, must diminish or grow in proportion to the possibility of these induced currents developing more or less freely. The influences producing the induced currents and governing their effects were shown to be manifold. It is quite conceivable that as one or the other of these influences (impact between adjacent particles, restriction in free area by eddies, repeated acceleration and retardation of every single particle) assumes a more or less prominent rôle, secondary maxima and minima of the total resultant loss should be formed.

To try to make any more definite statements would be rash, in face of the scant experimental evidence at hand; still, as this evidence covers a range of widths from 2 to 30 in., of ratios, $\frac{R}{d}$, from less than unity to 20, speeds from 1 to 10 ft. per sec., and degrees of roughness varying from smooth brass, through galvanized iron, to rusty cast iron, a network of interpolation can be thrown over the ground covered

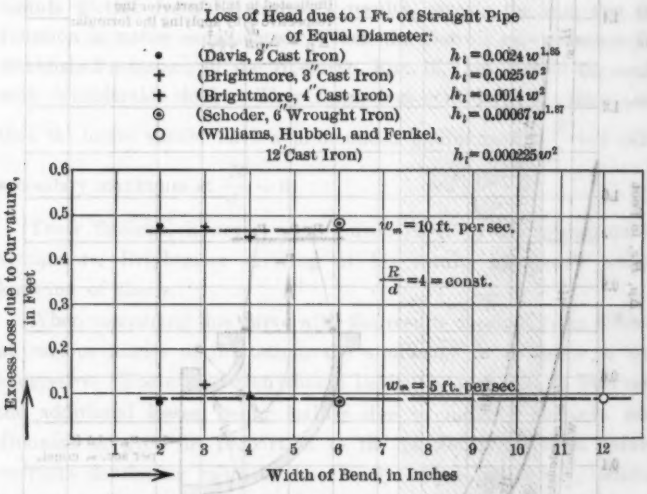


EXCESS LOSS OF HEAD DUE TO CURVATURE, H_e, AND LOCAL DEPRESSION AT OUTLET, Δp_B, AS MEASURED BY BRIGHTMORE ON 3 IN. AND 4-IN., RUSTY, COMMERCIAL ELBOWS, IN FUNCTION OF THEIR DEGREE OF CURVATURE, $\frac{R}{d}$.

FIG. 13.

by experiments. An effort was made by the writer by this means to put into the hands of the engineer a small comprehensive chart showing in a graphical way the range of values which the additional losses due to the curvature of U. S. Standard elbows might be expected to assume for various speeds of flow.

Davis succeeded in plotting a diagram, such as Fig. 14, based on the ratio, $\frac{R}{d} = 4$, from which it would appear that, with a fair degree



APPARENT INDEPENDENCE OF LOSS DUE TO INDUCED CURRENTS FROM WIDTH AND ROUGHNESS (G.J.DAVIS' CHART)

FIG. 14.

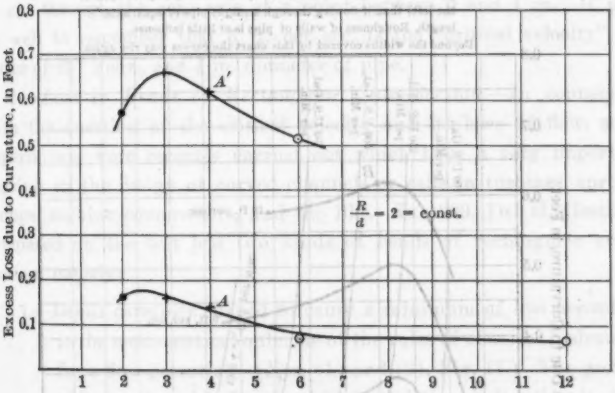
of approximation, the excess loss of head due to curvature was constant for the same speed of flow, irrespective of width and roughness of the bend. By carrying out similar graphic compilations of available data for other degrees of curvature (for example, Fig. 15), the writer became aware of the probability that this may not be a general rule. The variations of loss with increasing width, as emphasized by Fig. 15, naturally must be smallest for the ratio, $\frac{R}{d} = 4$, because there the total losses are a minimum.

The curvature of U. S. Standard elbows changes with the width. It is smooth for small, and rather abrupt for large, widths of pipe:

Diameter, $1\frac{1}{4}$ in., $\frac{R}{d} = 4$;

" 30 " $\frac{R}{d} = 1.8$.

By preparing diagrams such as Figs. 14 and 15 covering the range of the curvatures of standard elbows, and intentionally disregarding whatever influence of roughness there may be, the writer arrived at the values shown by Fig. 16. For example, a 4-in. U. S. Standard long-



SPECIMEN OF CHARTS ON WHICH FIG. 19 IS BASED

FIG. 15.

turn elbow has a curvature of $\frac{R}{d} = 2.01$ for which the excess loss may be found at the points, A and A', in Fig. 15, and transferred to the points, B and B', in Fig. 16, and so on. The addition of the cross-curves indicating the quantity of water passing through pipes of respective width at the given speeds might help to make the diagram handier for practical use.

Fig. 16 might also, without risk of great errors, be used in connection with 1912 Standard elbows as indicated, for their curvatures do not differ very much from those of the former and are smoother throughout.

The values, as plotted in Fig. 16, should be regarded as preliminary approximations, though based on a great number of very careful tests under actual conditions of use. It should be noted that the exact

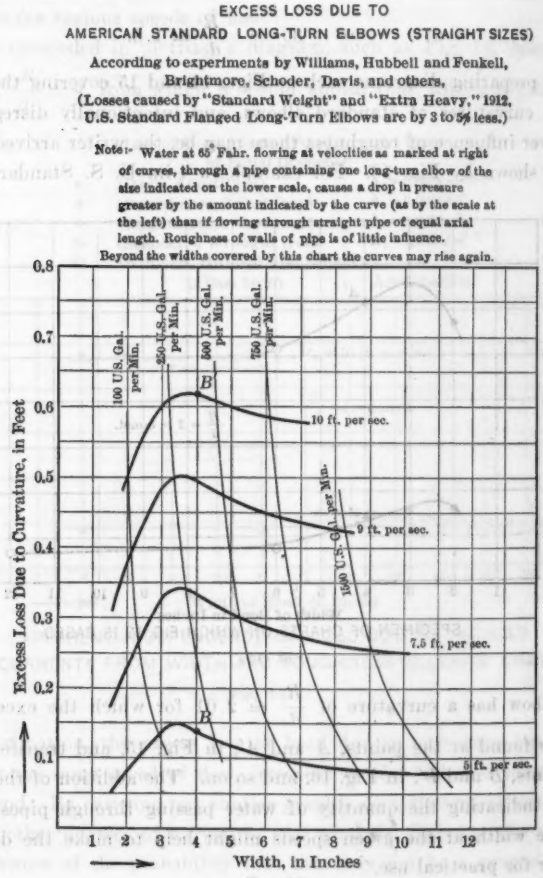


FIG. 16.

influence of roughness is by no means cleared up, and, when once definitely known, may alter somewhat the curves of Fig. 16, though the writer believes that their possible inaccuracy will by no means

be such as to endanger their usefulness seriously. The writer, in plotting these curves, believed that even approximate values would be more welcome than the present state of absolute uncertainty as regards these losses. It should be emphasized that the curves only hold good for water of ordinary temperature; further, that due provision (of about + 10%) should be made for the unavoidable inaccuracies in fitting pipes together at the joints. If the cross-section of an elbow is diminished by sediment, the actual speed of flow through the reduced area must be calculated and Fig. 16 consulted for the loss of head occurring at that speed.

It will be noted that the 5 ft. per sec. loss curve seems to tend to cut through the zero axis at a point between 0 and 1 in. It may be well to remember that 5 ft. per sec. is the "critical velocity" for water of 65° Fahr., and $\frac{3}{4}$ in. diameter of pipe.

b.—Loss in Bends of Rectangular Cross-Section.—In connection with the question of the critical velocity and its laws of flow, some experiments were recently carried out which have a very important bearing on the design of curved channels or cells in turbines, turbine pumps, turbine compressors, and the like. In 1909, Dr. H. Grether* compared on the test bed two kinds of bends of rectangular cross-section, namely:

1.—Bends carefully shaped to cause a minimum of loss according to the mathematical outcome of the rules of classical hydraulics for a flow-potential. (Nos. (1) and (2), Fig. 17.) The profiles given to these bends are equipotential hyperbolas, that is, they follow stream lines which according to theory the particles of an ideal fluid would describe when flowing at constant pressure and velocity around a corner.

2.—Simple bends of uniform cross-section forming a circular arc.

The expectation that the equipotential bends would cause the least drop in pressure of the compared variations of shape was fully realized as long as the speeds of flow remained below the critical limit. This was proved by using a viscous oil as a flowing medium.

As soon, however, as water was used for testing, and the critical speed was far exceeded, the results were a complete reversal of the first:

* "Ueber Potentialbewegung tropfbarer. Flüssigkeiten in gekrümmten Kanälen," Berlin, 1909 (Simion's Verlag).

For speeds of flow above the critical limit the equipotential bend caused the highest resistance to flow.

It is impossible, even roughly, to outline in this paper the ingenious details of Grether's remarkable investigations.

The writer only wants to emphasize, that if designers of hydraulic machinery would train their instinct for the correct shapes of blades or profiles by mere geometrical considerations, they would arrive at conclusions which are diametrically opposite to practical experience.

Parenthetically it might be remarked that, according to Grether's results, bends of rectangular cross-section offer a lesser resistance if—*ceteris paribus*—the cross-sectional rectangle stands on its small face (Nos. (3) and (4), Fig. 17) than if it lies on its broad side.

4.—Local Diminution of Pressure in Bends.

a.—Rectangular Cross-Section.—Pfarr used his experimental bend $\left(\frac{R}{d} = 1\right)$ for investigating the extent to which Equations (9) and (10) could be used for practical calculations. For this purpose, he measured at all speeds the minimum pressure at the inner wall. This generally occurred at a radius which was inclined against the entrance radius by about 50 degrees. He also measured the maximum pressure prevailing in the same radial cross-section. Table 2 is an extract from his data.

TABLE 2.—Comparison of Calculated and Measured Pressure Differences Forming at the Inner and Outer Wall of a Bend Having $\frac{R}{d} = 1$. (Denominations as per Equation (9).)

Calculated.				Measured.
v_m, in feet per second.	v_i, in feet per second.	v_a, in feet per second.	Δp, in feet.	Δp, in feet.
9.82	17.9	5.96	4.125	4.26
18.03	35.8	11.92	17.67	17.20
27.45	53.7	17.88	39.90	39.45

The two last columns tally remarkably well.

In addition, Dr. Lell corroborated these findings by a series of special tests which verified the expectation that the maximum and minimum pressures on the two walls (though by no means the pressure diagrams across any one radial cross-section) did, under all cir-

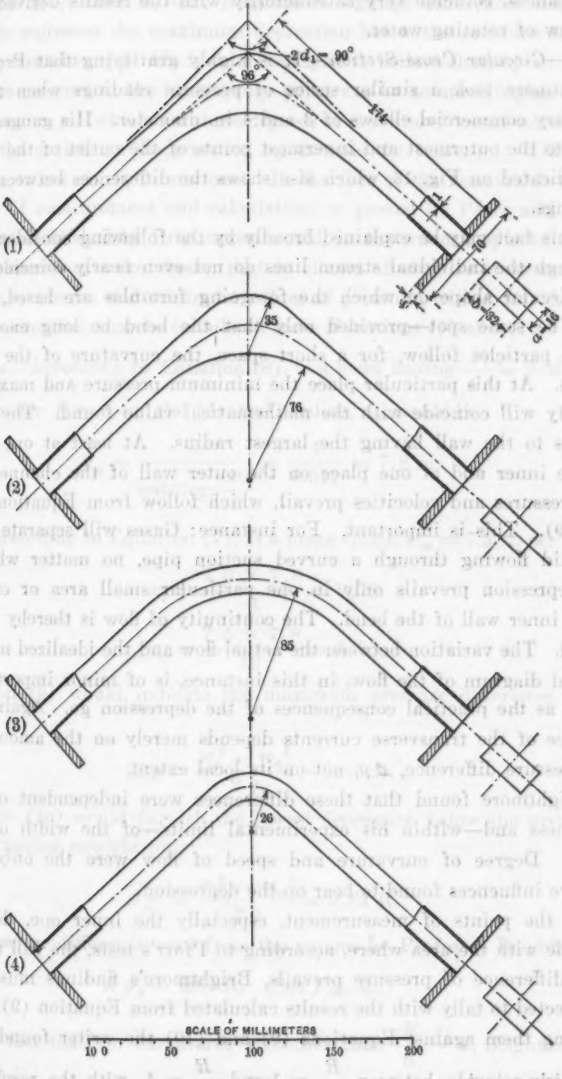

$2\alpha_1 = 90°$
96°
174
(1)
10
35
76
(2)
85
(3)
26
(4)
SCALE OF MILLIMETERS
10 0
50
100
150
200

FIG. 17.

cumstances, coincide very satisfactorily with the results derived from the law of rotating water.

b.—Circular Cross-Section.—It is highly gratifying that Professor Brightmore took a similar series of pressure readings when testing ordinary commercial elbows of 3 and 4 in. diameter. His gauges were fixed to the outermost and innermost points of the outlet of the elbow, as indicated on Fig. 13, which also shows the differences between their readings.

This fact may be explained broadly by the following consideration: although the individual stream lines do not even nearly coincide with the circular shape on which the foregoing formulas are based, there must be some spot—provided only that the bend be long enough—where particles follow, for a short space, the curvature of the inner radius. At this particular place the minimum pressure and maximum velocity will coincide with the mathematical value found. The same applies to the wall having the largest radius. At least at one place on the inner and at one place on the outer wall of the channel will the pressures and velocities prevail, which follow from Equations (6) and (9). This is important. For instance: Gases will separate from a liquid flowing through a curved suction pipe, no matter whether the depression prevails only in one particular small area or on the whole inner wall of the bend. The continuity of flow is thereby interrupted. The variation between the actual flow and the idealized mathematical diagram of the flow, in this instance, is of minor importance, as far as the practical consequences of the depression go. Again, the violence of the transverse currents depends merely on the amount of the pressure difference, Δp, not on its local extent.

Brightmore found that these differences were independent of the roughness and—within his experimental limits—of the width of the elbow. Degree of curvature and speed of flow were the only two decisive influences found to bear on the depression.

As the points of measurement, especially the inner one, do not coincide with the area where, according to Pfarr's tests, the full maximum difference of pressure prevails, Brightmore's findings must not be expected to tally with the results calculated from Equation (9). By checking them against Equations (9) and (10) the writer found that they fairly coincide, between $\frac{R}{d} = 1$ and $\frac{R}{d} = 4$, with the results of

Equation (10), that is, that Brightmore's gauge differences may be taken to represent the maximum depression below the pressure of the ingoing straight pipe. He feels encouraged to believe that, by an accident, the pressure at the inner gauge point was by so much higher than the minimum p_i (which, for these degrees of curvature, occurs somewhat before the outlet) as the pressure, p_a, exceeds the value, p_m. This, by the way, is not much, in any case. In the face of the remarkable coincidence of measurement and calculation, as proved by Pfarr and Lell, the data established by Brightmore would, in the writer's opinion, only tend to corroborate the thesis: that in elbows of circular cross-section, too, the maximum and minimum values of pressure, with a fair degree of approximation, may be calculated from Equations (9) and (10). For example: According to Equation (8), in a bend having $\frac{R}{d} = 2$, $d = 4$ in. diameter, $R = 8$ in., the "neutral" radius, r_m, would be:

$$r_m = \frac{4}{\text{nat. log. } \frac{10}{6}} = \frac{4}{0.511} = 7.8 \text{ in.}$$

Then, according to Equation (7), at a mean speed, $w_m = 7.5$ ft. per sec.:

$$w_i = \frac{7.5 \times 7.8}{6} = 9.75$$

$$w_a = \frac{7.5 \times 7.8}{10} = 5.85.$$

Equation (9) would indicate the maximum pressure difference, Δp, to be:

$$\Delta p = \frac{w_i^2 - w_a^2}{2g} \sim 0.95 \text{ ft.}$$

Equation (10) would furnish the lowest depression below the pressure in the ingoing straight pipe:

$$\Delta p' = \frac{w_i^2 - w_m^2}{2g} = 0.61 \text{ ft.}$$

The pressure difference read from the gauges by Professor Brightmore was:

$$\Delta p_B = 0.64 \text{ ft.} \quad \text{(See Fig. 13.)}$$

The fact that, for curvatures smoother than $\frac{R}{d} \sim 4$, Brightmore's figures show a rising tendency is, in the writer's opinion, due to the

circumstance that the position of the actual minimum pressure, p_i, shifts more and more toward the point where his instruments were fixed, namely, immediately in the outlet cross-section. This would mean that, for $\frac{R}{d} < 4$, he measured a difference of pressure smaller than Δp, and, for $\frac{R}{d} \sim 10$, he measured the full amount, Δp, as defined by Equation (10). Indeed, the value of Δp_B found by Brightmore for $\frac{R}{d} \sim 10$, namely, about 0.22 ft., tallies tolerably well with—and even exceeds—the value, Δp, as calculated from Equation (9), namely, 0.19 ft.

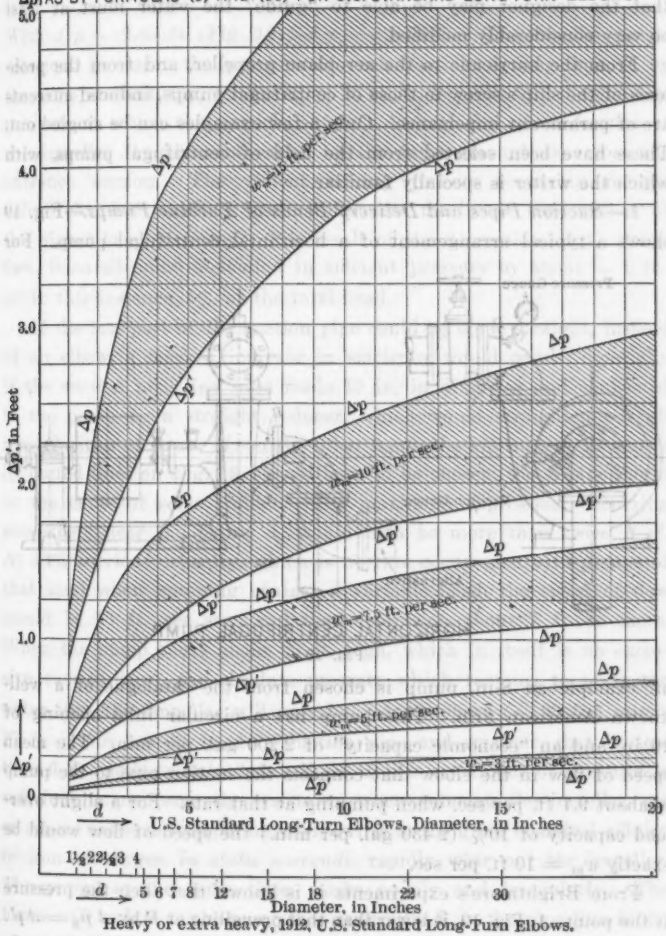
It can be taken as granted beyond dispute that depressions as indicated by Equation (10) (in combination with Equations (7) and (8)) do actually occur. It matters little at present whether they are 20% larger or smaller than the results derived from Equation (9). What does matter is that depressions are formed which are a multiple in many cases (and especially in standard elbows of large diameter) of the loss of head caused by the elbow. In order to supply the designer of pipe lines and hydraulic machinery with a handy means of estimating what depressions, $\Delta p'$, and maximum differences of pressure, Δp, must be expected, the writer has developed from Equation (10) the diagram, Fig. 18, which speaks for itself. It is again based on the curvatures of U. S. Standard long-turn elbows. As only the ratio, $\frac{R}{d}$, matters (besides the speed of flow), a second scale for the abscissas was added, having reference to 1912, U. S. Standard long-turn elbows. When comparing both abscissas it is evident that the slightly greater smoothness in curvature which was introduced with the 1912 Standards serves to diminish considerably the dangerous depressions which form in the elbow.

It is almost needless to emphasize the fact that short-turn elbows and T-pieces must cause very much more severe depressions. Therefore, they should never be allowed in suction pipes for great suction heads and high rates of flow.

III.—Some Special Practical Results.

No changes in the direction, and no retardation, of flow are possible without the generation of induced currents. They influence the flow

CALCULATED DEPRESSIONS, $\Delta p'$, AND MAXIMUM PRESSURE DIFFERENCES, Δp, AS BY FORMULAS (9) AND (10) FOR U.S. STANDARD LONG-TURN ELBOWS.



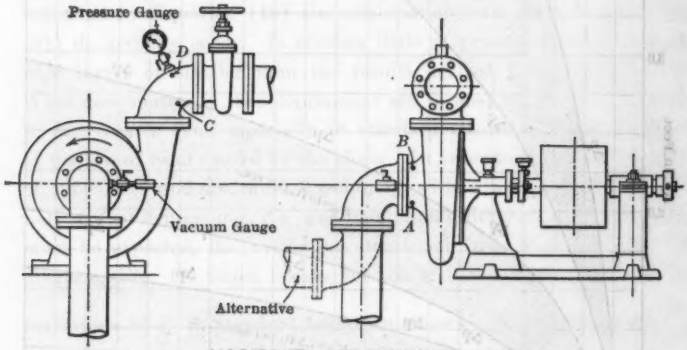
Heavy or extra heavy, 1912, U.S. Standard Long-Turn Elbows.

FIG. 18.

of fluids in such a way that in no case does the path on which the drops travel coincide with assumed "stream lines". The water flows through turbines and pumps on complicated spiral paths. The idea that the designer may be able to "guide" the water must at least be very considerably modified.

From the hurricane to the aeroplane propeller, and from the problems of the ship's screw to those of centrifugal pumps, induced currents are of paramount importance. Only a few examples can be singled out. These have been selected from the field of centrifugal pumps, with which the writer is specially familiar.

1.—Suction Pipes and Delivery Bends of Turbine Pumps.—Fig. 19 shows a typical arrangement of a horizontal centrifugal pump. For



HORIZONTAL CENTRIFUGAL PUMP

FIG. 19.

an example an 8-in. pump is chosen from the catalogue of a well-known American firm. This pump has a circular inlet opening of 10 in. and an "economic capacity" of 2 200 gal. per min. The mean speed of flow in the elbow that connects the suction pipe to the pump is about 9.1 ft. per sec. when pumping at that rate. For a slight overload capacity of 10% (2 430 gal. per min.) the speed of flow would be exactly $w_m = 10$ ft. per sec.

From Brightmore's experiments it is known that then the pressure at the point, A, Fig. 19, is lower than that prevailing at B by $\Delta p_B = \Delta p'$. The value of $\Delta p'$ is found, from Fig. 18, to be, for a 10-in. U. S. Standard long-turn elbow at $w_m = 10$ ft. per sec.:

$$\Delta p_B = \Delta p' = 1.65 \text{ ft.}$$

The actual depression, Δp_a, at the point, A, below the pressure of the "neutral vein", may then be estimated by applying the ratio:

$$\Delta p_a = \Delta p_B \frac{\Delta p'}{\Delta p}.$$

With $\Delta p \sim 2.35$ ft. (Fig. 18), $\Delta p_a \sim 1.15$ ft. The impeller, by its rotation, increases the pressure of each particle flowing into it by the same amount, say, for example, 20 ft., independent of the absolute pressure at which the particle enters. Around the circumference of the impeller the pressure varies by the same amount as across the entrance section. The violent mixing process occurring in the volute chamber equalizes these differences mostly at the expense of the higher local pressures. The result of the depression, Δp_a, therefore, is an all-round reduction in efficient pressure by about ~ 1 ft., or, in this instance, 5% of the total head.

If the last part of the suction pipe could be made straight, instead of an elbow, a marked increase in efficiency would result, especially if the suction pipe line were made 12 in. in diameter and connected to the pump by a straight reducer, which would straighten out all irregularities of flow. If, on the other hand, the alternative, as shown by dotted lines on Fig. 19, should have to be chosen, the **S**-pipe might be the cause of some trouble. The maximum depression, occurring somewhere near the point, A, would then be more than twice $\Delta p'$. At 30% overload capacity, which is no rare occurrence with pumps of that kind when pumping against a variable head, the speed of flow would be 13 ft. per sec., and that depression about 7 ft. or more. When the pump sucks about 20 ft. high, which in itself is no excessive suction head, the absolute pressure which remains to press the water into the impeller will only be: Atmosphere ($\sim$ 33 ft.) minus 27 ft. minus loss of head in suction pipe and strainer, that is, less than 6 ft., perhaps only 3 ft. Even if this remainder of pressure is sufficient to overcome the resistance when entering the impeller, air or vapors are sure to separate, and, owing to the great chemical affinity to iron of oxygen *in statu nascendi*, rapidly wear out the impeller. The pump, at the same time, works noisily and irregularly. The blame will be laid to the pump, but really the **S** in the pipe line is the cause of all the trouble.

In case trouble arises from the decrease in head or efficiency owing to the effect of the elbow leading into the pump, a smart representative

of the pump manufacturer might easily prove that "nothing is wrong with the pump" by fixing a pressure gauge at the point, D (Fig. 19), and a vacuum gauge midway between A and B. The latter will then indicate approximately the real mean entrance pressure (pressure of "neutral vein"), while the pressure gauge reads too high by the amount, $\Delta p - \Delta p'$. Taking, again, 2 430 gal. per min. as the rate of flow, then $w_m \cong 15.5$ ft., and, by aid of Fig. 18, or Equations (9) and (10):

$$\Delta p - \Delta p' \cong 1.9 \text{ ft.}$$

for an 8-in. Standard elbow. If the total head is low, the capacity large, and the elbows sharp, the importance of such an error is obvious. Either several gauges should be fixed all around the delivery orifice or the reading should be taken on a straight pipe after the water has passed a straight reducer, making allowance, of course, for the increase in velocity head entailed thereby.

2.—Impellers.—Closed impellers consist of a number of cells formed by the profiles of the impeller and by the back of one and the front of the next following blade. They are curved in two planes: first in the plane which contains the axis of the pump (curvature of profile); secondly in the plane rectangular to the former (curvature of blades).

When trying to apply the results gained from stationary bends to the cells of impellers, insuperable difficulties arise. These are specially great with reference to the curvature of the blades. In addition to internal and external friction of the fluid, three components of force are acting: one due to the centrifugal force of rotation, one due to the acceleration of the fluid relative to the impeller, and one due to the acceleration of Coriolis, that is, supplementary acceleration due to composite motion. It seems improbable that, within the near future, a clear and simple mathematical expression could be found covering the effect of the co-operation of these five component forces. To be of immediate practical use, the mathematical treatment would have to be simple enough to be applicable economically in the designing office of a factory. This possibility seems to be very remote.

The only benefit which designers may reasonably hope to derive from such mathematical and experimental research work is a development of a sound judgment for the constructive shapes necessary to produce or avoid certain effects. After all, this is the principal

basis of all industrial production of machinery: an engineering instinct based on the results of engineering science or physics.

When looking for such benefits, the application of the knowledge of induced currents to the curvature of profile is bearing some important fruits. The component forces due to rotation and to the acceleration of Coriolis are negligible around the eye or hub, especially where it is well machined. Within the cells proper their action is confined principally to planes parallel to the profile, and, therefore, should not be of very considerable influence on the diagram of pressures along the axial plane. The effects of the curvature of profile, therefore, may well be expected to be similar to those produced by the fluid streaming at an equal relative velocity through the stationary cell. The writer contends that, whatever the indirect effects of the two other components of external forces may be, the cause responsible for the "law of the rotating water", that is, the inertia of the relative flow, does not disappear. Its effect must be discernible in the final diagram of energy at the circumference of the impeller.

As the conclusions which the writer reaches on this base are verified throughout by practical experience, he ventures to outline them here in the hope that discussion may help to bring forth all available evidence of experiment and experience, either to support or to refute his contentions.

Fig. 20 represents a comparison of two extreme types of impellers. Each has the same axial length over all (l). The free entrance areas of the impellers (D'_1 diameter) and the individual cells shall be equal in both instances, also the angle, β_2, between the blades and the periphery, and the free discharge areas at that spot. The axial width, b_2, at the periphery, consequently, must be inversely proportional to the diameters, D_2. If the same quantity is delivered by each impeller, the radial components of relative speed, v (that is, those lying in the drafting plane), are equal throughout. The left-hand impeller is a typical low-lift, the right-hand one is a high-lift runner. If both rotate at an identical number of revolutions per minute, then the total heads generated are, in a rough approximation, proportional to the square of the diameter, D_2. The same holds good for the static pressures, *H st.*, prevailing at the circumference, which constitute an equal percentage of the total head in both cases, as the angles, β_2, are the same. Now, the differences in pressure, owing to the law of

rotating water, must influence the disposition of the peripheral pressure across the width of the impeller in the manner indicated by the curves, A' B' and A'' B''.

The ratio between width of cell and radius of curvature of the profile automatically turns out more favorable with the high-lift impeller. Furthermore, in a high-lift runner, the profile usually runs straight for a considerable distance before reaching the periphery. Both advantages result in the difference, $H_{max.} - H_{min.}$, being much

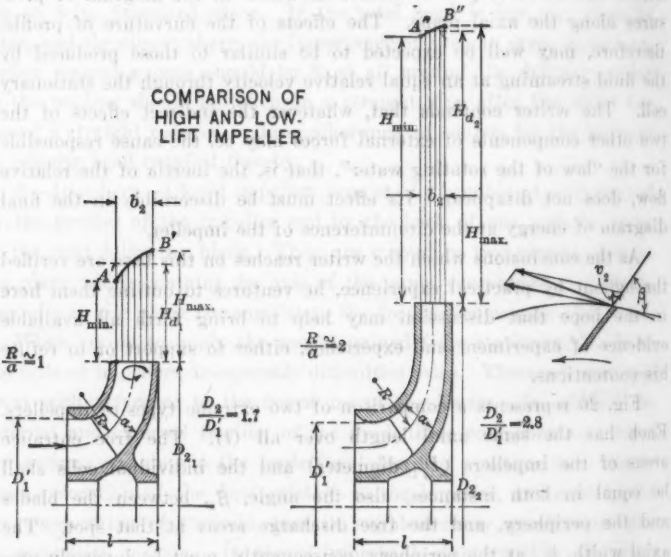


FIG. 20.

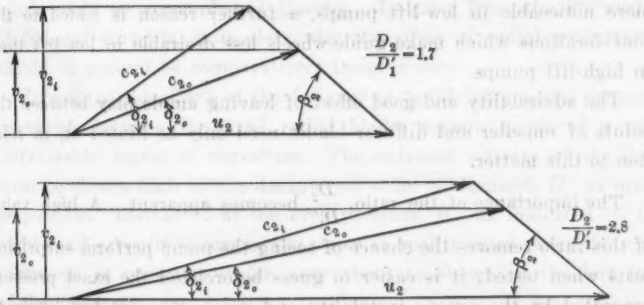
smaller absolutely in high-lift impellers. Relatively, the comparison is still more in favor of the high-lift runner. In Fig. 20, $H_{d_1} : H_{d_2} \cong D_{2_1}^2 : D_{2_2}^2$, consequently, an induced current of considerable energy, as characterized by the arrow, must influence, and in extreme cases even jeopardize, the generation of useful pressure in the low-lift impeller, though in high-lift impellers this influence is generally negligible.

A secondary consequence is a difference of pressure in the left-hand and right-hand clearance spaces at the periphery, which adds to the axial thrust an unknown quantity. This component of the axial

thrust does not vary with the head, but evidently with the square of the relative velocity, that is, of the delivery, and it increases when the head decreases. This adds one more difficulty to the problem of balancing the axial thrust, especially in low-lift pumps.

Correspondingly, the velocity diagram at the periphery is influenced by the secondary components. The higher pressure corresponds to a lower velocity and *vice versa*. The effect is suggested in Fig. 21. Again, the variation of the velocity diagram is enhanced by the following peculiarities of the low-lift runner:

a.—The unfavorable degree of curvature, $\frac{R}{d}$, entails a great difference between the radial component, v_2, of the relative velocity at the inner (v_{2_i}) and outer (v_{2_o}) side of the bend.



VARIATION OF VELOCITY ACROSS WIDTH OF IMPELLER AT PERIPHERY.

FIG. 21.

b.—The absolute discharge velocity, c_2, is influenced more by v_2 when the circumferential velocity u_2, is comparatively small, that is, when the ratio, $\frac{v_2}{u_2}$, is high.

c.—The angle, δ_2, between absolute (c_2) and peripheral (u_2) velocity, again varies considerably more if $\frac{v_2}{u_2}$ is high.

The fluid leaves the impeller under a steeper angle, δ_2, at the left side than at the right. As the guide- or diffusor-blades are without exception made cylindrical, that is, curved in only one plane, it is impossible, on principle, that at any duty all threads of fluid should

at all points enter the guide-channels parallel to the guide-blades, that is, "without shock", as the term is used.

The loss due to this cause will reach a minimum when the angle of the guide-blades is somewhere between the two extremes of δ_2, as suggested in Fig. 21. As velocities at this point are very considerable, the efficiency, especially of large medium-lift pumps, can be appreciably improved if the angle of the guide-blades is not based on the usual diagram of discharge velocities, but on such a diagram as Fig. 21, choosing a value for α_3 somewhere between the two extremes. Experiments conducted by Messrs. Sulzer Brothers, in Winterthur, Switzerland, with a view to determining the influence on the efficiency of such variation of δ_2, have shown that this point well deserves the attention of designers.

As the lack of parallelism of the water threads is generally much more noticeable in low-lift pumps, a further reason is added to the considerations which make guide-wheels less desirable in low-lift than in high-lift pumps.

The advisability and good effect of leaving ample play between the points of impeller and diffusor blades need only be hinted at, in relation to this matter.

The importance of the ratio, $\frac{D_2}{D_1'}$, becomes apparent. A high value of this ratio removes the chance of seeing the pump perform surprising feats when tested; it is easier to guess beforehand the exact pressure generated by the pump; instability and noise, etc., are less likely to occur unexpectedly. On the other hand, difficulties are bound to arise if a runner with a small ratio of diameters, for instance, $\frac{D_2}{D_1'} \leqq 1.5$, has to suck the water through a sharp bend as shown on Fig. 19. The depression forming at the point, A, Fig. 19, serves to enhance the depression at A', Fig. 20. The induced current becomes so strong that a complete vortex may be formed at times, resulting in a surprising reduction in capacity, instability of action, noise, lack of efficiency, etc.

The designer's influence on the determination of that decisive ratio, $\frac{D_2}{D_1'}$, is rather limited. The entrance diameter, D_1, is either fixed by standardized manufacture or, at all events, dependent on the capacity. D_2 is given by the required head and speed of the pump. Espe-

cially the problem, so prominent at present, of coupling water-works. irrigation, or drainage pumps direct with high-speed motors or steam turbines nearly always places the designer in a quandary. The head is low and the speed high; therefore D_2 is very small. The capacity is great; therefore D_1' is large. The expedient of reducing D_1', by splitting the capacity and coupling two or more impellers in parallel, generally is barred by competitive reasons, for this means a considerable increase in weight, selling price, and floor space required by the pump.

The outcome of such a proposition is shown by the upper half of Fig. 22, which is taken from a publication of a well-known American manufacturing concern. In order to prevent instability of flow, an elaborate system of radial entrance guide-vanes, A, was introduced. From the writer's experience it would seem to be doubtful whether much could be accomplished thereby. Judging from the knowledge of induced currents, it is clear why the effect of radial guide-vanes should in general be comparatively insignificant.

In spite of the efforts of the designer to "guide" the water, the compartments between the fixed radial blades remain bends of a very unfavorable degree of curvature. The entrance velocity of the fluid must be chosen high by the designer in order to diminish D_1' as much as possible. Instead of at the even pressure, H_1, as indicated on the right-hand pressure diagram, the water in reality must enter the compartments at pressures which vary about as shown in the left half of the diagram.

Worse still is the corresponding variation in speed, which results in heavy losses through discrepancy of angles at the entrance of the impeller. Until the water reaches the circumference of the impeller, this difference must be further emphasized. The resulting mean pressure, H_{2_m}, may even drop below the expected amount, H_2. In any event, the water will leave the impeller in a greatly disturbed condition, which will not add to the efficiency of the pump.

The depression at the point, P, may even lead to a separation of air or vapors. From the knowledge of induced currents, a suggestion might be made which would seem better to serve the purpose and which is sketched in the lower half of Fig. 22. This arrangement does not require more space than the former. By introducing an intermediate profile, C, the degree of curvature is improved very consider-

ably, for both of the newly formed halves of the runner. In order fully to utilize this advantage, the inlet chamber, too, should be divided in two by a cylindrical guide, A'. Though, no doubt, the impeller will become a more complicated casting and, therefore, more expensive

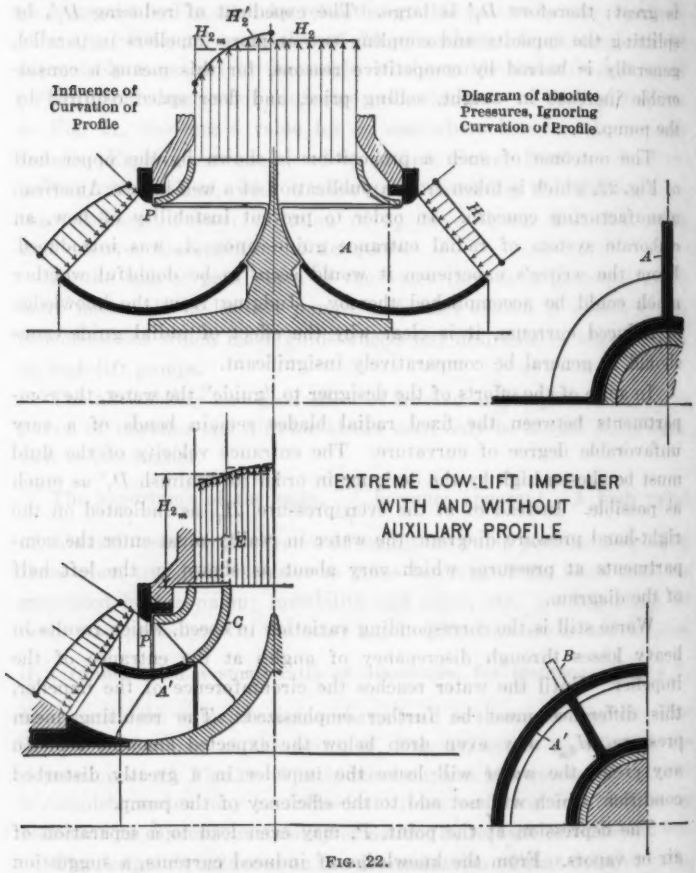


FIG. 22.

in manufacture, the new guide is considerably smaller and cheaper than the radial guide-blades, A, shown in the upper half of Fig. 22. Exterior radial ribs (as shown by the dotted lines at B on the lower right cross-section) may or may not be resorted to, in addition to the

cylindrical guide, A'. By doing away with them in the outer half of the inlet throat (as shown in the lower left section), the advantage is secured of having the water rotate somewhat around the axis of the pump, owing to the friction of the impeller. This counteracts the depression which forms at the point, P. Incidentally, in the arrangement shown in the lower half of Fig. 22, the passage area of the throat is made steadily to decrease, thus securing the steadying effect of a nozzle for straightening out irregularities of flow at the entrance of the impeller.

The result of this and of halving the width of the cells is suggested in the pressure diagram erected over the width of the impeller outlet. The extreme pressure differences should be reduced in both halves, as compared with the arrangement shown in the upper half of Fig. 22. The danger of a break in continuity is substantially removed.

A new difficulty arises: both the absolute and the mean pressures are different on both sides of the auxiliary profile. In addition, the direction and speed of flow will differ at this place. Dr. Lell's experiments tend to show that the eddies which must result therefrom do not nearly absorb the increase in final pressure which actually does occur by inserting an auxiliary profile. Besides, it will often be possible, without great difficulty or complication, to provide for a short auxiliary wall, E, which keeps the two streams apart until their velocity has decreased sufficiently.

An obvious objection to the suggested alternative will be directed against the increase in loss of head which should result from the additional area of friction introduced by the intermediate partition, or, in other words, from the diminution of the hydraulic radius. No doubt the auxiliary profile does increase that part of the loss of head which is directly due to wall friction. The losses due to the induced currents, however, which are a multiple of the friction losses, are reduced. Their reduction is of much more importance than the increase in wall friction.

That this contention is correct was proved by Dr. Lell. He inserted in Pfarr's experimental bend an auxiliary partition forming a concentric cylinder, of 170 mm. radius, and of the same smoothness or roughness as the walls of the bend. Though, by this insertion, the total area of the walls was increased by 26%, the loss of head in the bend proper decreased by 20 to 25 per cent. In Fig. 12 the curve

of dashes and dots, marked, "With partition (equalized)", furnishes a means of comparing the losses occurring with and without such partition.

Lell's investigations also proved the advantageous effect of continuing the partition some little distance into the straight duct following the bend. For these and similar details the writer must necessarily refer those interested to Lell's paper.

It should be emphasized that the advantage to be derived from an auxiliary partition in the impeller is not only the decrease in loss of head through the curvature of the impeller profile. This, in itself, would not amount to very much. If 5% of the total energy transformed in a turbine pump were lost in the impeller proper, a reduction of such losses by 20% would only improve the efficiency by 1 per cent.

The deviation of actual velocity diagrams at the entrance and discharge of the impeller from the theoretical diagram is much lessened by the introduction of the auxiliary partition. This fact should influence the efficiency, directly and indirectly, much more favorably than the mere reduction of impeller resistance.

3.—Diffusor Vanes.—It is difficult to derive more than a few general points of view regarding guide-wheels of turbine pumps from the experimental data which are available at present. Gibson, in a noteworthy paper,* has put down a few suggestions as to how, on the basis of his experimental evidence, the channels of guide-wheels could most advantageously be shaped. The writer would refer those interested to that paper.

The fact that, in most instances, the axis of the guide-channel must, from constructive reasons, be curved concavely toward the impeller, gives a special importance to the curvature of the outer wall (o — o in Fig. 23) of the channel. Some designers are of the opinion that, in multi-stage pumps, the fluid should be "gradually guided into a radial direction before being led back from the periphery of the casing to the inlet of the next stage". The object is to avoid the formation of a vortex in the chamber, C. They, therefore, favor a change in the sense of curvature of the wall in question, about as shown by the dashed curve in Fig. 23. This is not good practice. The water refuses to be "guided". Instead, the abrupt opening out of the cross-section connected with the change of curvature often generates strong

* *Engineering*, February 16th, 1912, p. 205.

and obnoxious induced currents which seriously impair the efficiency, at least in all instances where the velocity of the fluid is still high at the turning-point of the vane. On the other hand, there is no danger in tangential forces which are exercised by the streams leaving the guide-channels, as long as radial ribs, *D*, are provided for in the chamber, *C*. These are not to be dispensed with in any event, even if the guide-channels "pour out radially"; for there is always the twist due to the induced currents developed in the curved guide-channels.

This "twist" or corkscrew motion is also present in the streams of water which leave the impeller. Andres' investigations on the effect

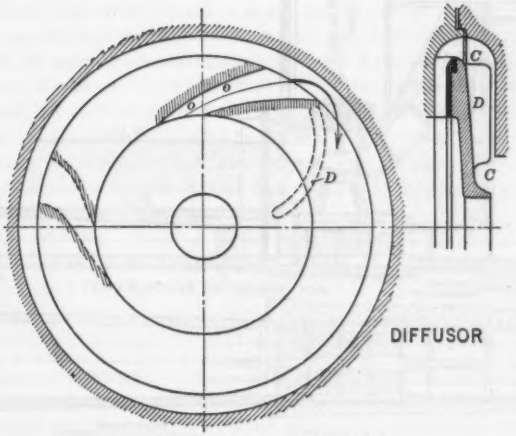


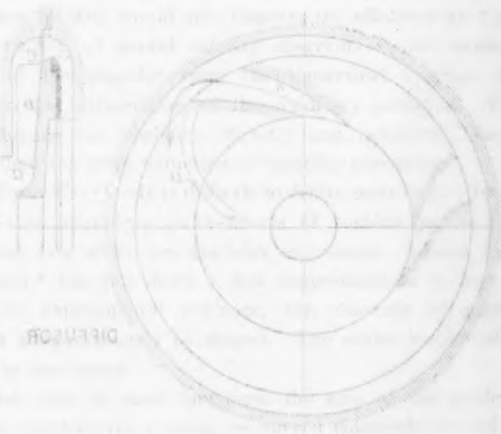
FIG. 23.

of rotation of the fluid around the axis of diffusors indicate that it is due to this spiral motion that the efficiency of diffusor channels in pumps seems to be tolerably high, in spite of the odd shape which the designer generally has to give them.

By the foregoing suggestions the writer feels that he is just skimming the possibilities of improvement based on a fuller knowledge of the induced currents of fluids. A fair proportion of this knowledge is at hand. Designers of hydraulic and pneumatic machinery should at least be able to derive some valuable aids from it to induce them to slacken their hold on the theory

of stream lines and to develop a wider and more correct conception of the actual flow of fluids. This cannot but lead to very substantial improvements in many directions.

Meanwhile, it will be incumbent on technical physics to supply more specific information about all details connected with the generation of induced currents of fluids.



DISCUSSION

Mr. Herschel.

CLEMENS HERSCHEL,* M. AM. SOC. C. E. (by letter).—A special case of induced currents of fluids is shown by the currents flowing into the throat of a Venturi tube from the outside, when thus allowed to flow in by suitable orifices. The writer has experimented on such currents on an unusually large scale, these experiments being described in a paper entitled "The Fall-Increaser", in the *Harvard Engineering Journal* of June, 1908. This is an account of experiments made to establish the

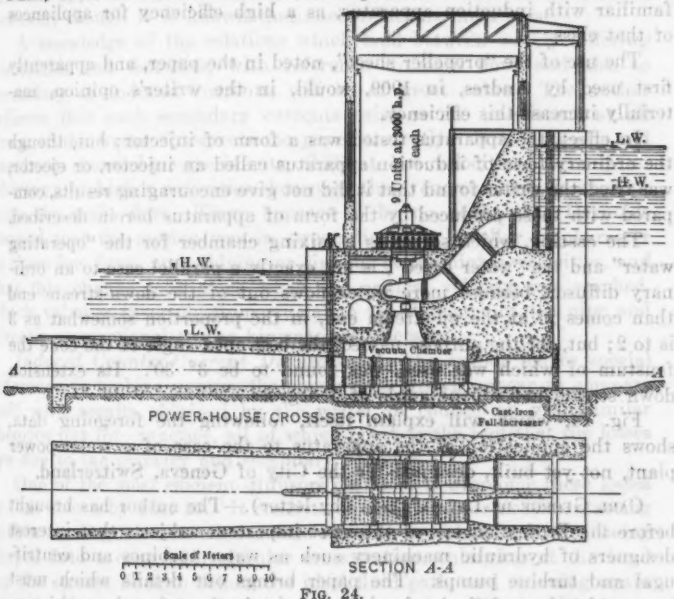


FIG. 24.

proper proportions of an apparatus called the "fall-increaser", designed to utilize, for power purposes, freshet water otherwise going to waste over the dam, in the case of water-power plants, on rivers having a suitable régime, when situated adjacent to, or not far distant from, the dam.

As the *Harvard Engineering Journal* is rarely seen, the results of the experiments alluded to may properly be recorded in conjunction with those referred to in the paper.

The experiments were conducted accurately at the Holyoke, Mass., public testing flume, all the water experimented with being measured

* New York City.

Mr. Herschel. continuously with two Venturi water meters. The "operating water" was in some of the tests as much as 14 sec-ft.; the "water lifted" (entering the tube by induced current), 7 sec-ft.; the natural, or "operating head", up to 14.35 ft.; the "head gained by the use of the fall-increaser", up to 12.52 ft.; and the penstock, 16 in. in diameter. With this apparatus, a vacuum, measured by more than 26 ft. of water column, could be produced and steadily held suspended; its efficiency, reckoning for water lifted a certain height, by water falling a certain height, as a maximum found, was 30.4%, which will be recognized, by those familiar with induction apparatus, as a high efficiency for appliances of that class.

The use of the "propeller sheet", noted in the paper, and apparently first used by Andres, in 1909, would, in the writer's opinion, materially increase this efficiency.

In effect, the apparatus tested was a form of injector; but, though the ordinary form of induction apparatus called an injector, or ejector, was tried, the writer found that it did not give encouraging results, compared with those produced by the form of apparatus herein described.

The casting, which serves as a mixing chamber for the "operating water" and the "water lifted", is not exactly a parallel case to an ordinary diffusor, because more water flows out of the down-stream end than comes in at the up-stream end, in the proportion somewhat as 3 is to 2; but, for the purpose named, the best apex angle of the cone the frustum of which was used, was found to be 3° 30′. Its extension down stream had a cone angle of 5 degrees.

Fig. 24, which will explain itself, following the foregoing data, shows the application of the apparatus to the case of a water-power plant, not yet built, designed for the City of Geneva, Switzerland.

Mr. de Laval. CARL GEORGE DE LAVAL,* ESQ. (by letter).—The author has brought before the Profession one of the most important subjects that interest designers of hydraulic machinery such as water turbines and centrifugal and turbine pumps. The paper brings out details which must be considered carefully in the design and selection of such machinery, and have only been known to a few within comparatively recent years. It is safe to say that a great many hydraulic machines have been designed by the purest guesswork, but, owing to the call for increased efficiencies and economy in operation in all the applications to various plants, the problems have received careful and intelligent study, causing many improvements, with more to come, when such papers as this bring them before the Profession for further thought.

It is admitted that improvements are hampered by the scarcity of published data from experiments, and, therefore, such a collection of

* Gen. Mgr., Worthington Pump Co., Ltd., Harrison, N. J.

data as the author presents will give valuable information for the use of manufacturers. Mr. de Laval.

In the evolution of centrifugal and turbine pumps no factor has played a greater part in obtaining increased efficiency than the study of the laws of the surface friction of fluids. This subject has been under investigation by the most eminent engineers for more than 100 years, and to-day the laws governing the velocity of flow for a particular fluid are tolerably well known. Experiments establishing a general relation applicable to all fluids and conditions of flow are not known, however, though it is known that such relations must exist.

A knowledge of the relations which hold between widely differing viscosities and densities, with reference to main flow, and also to secondary or induced currents, will be of great value. The author believes that such secondary currents exist or are co-existent at all flows, if the writer interprets the paper correctly. It is believed, and has been shown by experiments, that, at low velocities, there is no eddying; at higher velocities, however, these currents are co-existent with the main flow. There appears to be no doubt that Reynolds showed that the motion was in stream-lines, or lamellar in character at these low flows, and eddying or sinuous at the higher velocities, and that this change took place when the critical velocity was reached, which value is inversely proportional to the diameter of the pipe and directly proportional to the kinematical viscosity.

Induced Counter-Current Diffusors.—As the author makes special reference to turbine pumps, the conclusions drawn appear to compare well with results obtained by the writer in connection with similar theories put into practice, all of which point to the fact that the losses are due to the primary and secondary currents.

One of the most efficient diffusors designed to overcome these losses is that shown by Fig. 25. It consists of eight diffusor nozzles, starting with a rectangular section near the impeller periphery and then being shaped into a cone with circular cross-section, enlarging at the bend for the stream lines to follow the ideal fluid path, and reducing again to a circular section for the next impeller supply. This bend is circular, and will show a difference of pressure between the outer and inner points, due to the centrifugal force of the water flowing through them. This difference can be ascertained. The result is a vortex set up at the inner surface, which can be eliminated when the water is swept clean, due to secondary currents which also set up a rotary motion while passing through the diffusors.

The discharge from these eight diffusors is exactly the same as that from eight nozzles discharging parallel to the axis of a turbine pump. This arrangement appears to bear out the author's theory, and was applied by the writer on some important installations of turbine pumps with 250 ft. per stage, and gave a remarkable efficiency of

Mr. de Laval.

86 per cent. There was every indication, after the experiments were made with diffusors of this shape, that they gave a diffusion efficiency as high as 99.5%, and fully substantiated the proofs of Andres' experiments.

Transverse Induced Currents.—In considering this subject the writer believes that there are two critical velocities which must be taken into account: one is the velocity due to the fluid entering with a high state of turbulence, passing from an eddying to a stream-line motion; the other is that in which an undisturbed fluid enters with a

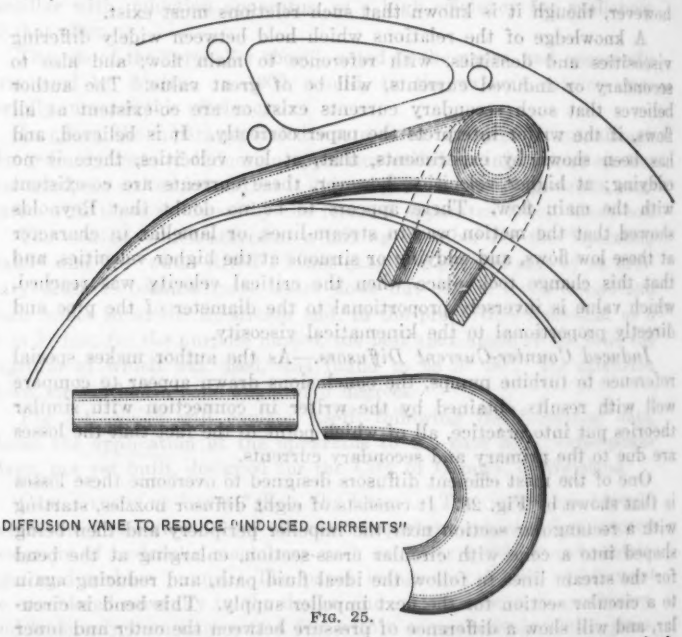


FIG. 25.

stream-line motion which exists until the critical velocity is reached before breaking into an eddying motion. This is just what takes place in elbows, bends, and suction or entrance heads to centrifugal pumps. The theory, after mathematical survey by the author, appears to give a good insight into the subject, with, perhaps, such modifications of the position of the elbow as relate to a consideration of the rotation of the impeller which is to receive this water finally.

Fig. 26 shows a design of the suction entrance of a pump having one turn or elbow for the water. This embodies the theory of gradually increasing the velocity in order to produce a vortex at the impeller

hub. It is to be noted that the upper section is divided like a heart, and stream lines are directed toward a cone at the impeller entrance. Mr. de Laval.

Fig. 27 shows a design of an entrance intended to obviate these induced currents, where the water has to enter horizontally at right angles to the rotation, and is properly fed or directed to the runner. Owing to the fact that the water receives a rotary motion, and that

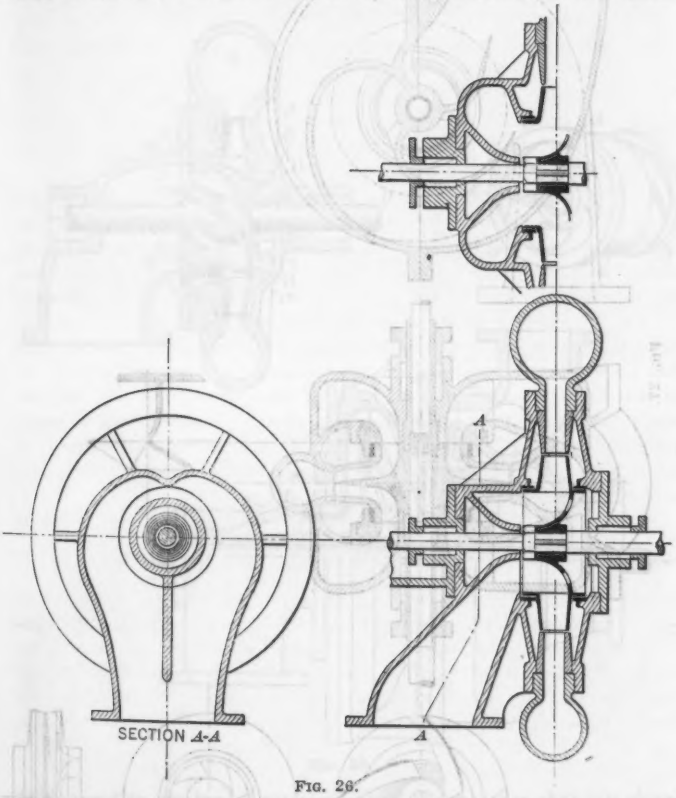


FIG. 28.

in this case there are a great many induced currents, the inflow of water to the runner would be greatly impaired and probably reduced considerably. This was shown by experiments similar to von Cordier's tests indicating by pressures at various points the various areas and places where these induced currents took place; they also revealed the fact that it was necessary to prevent the whirling of the particles in an

Mr. de Laval.

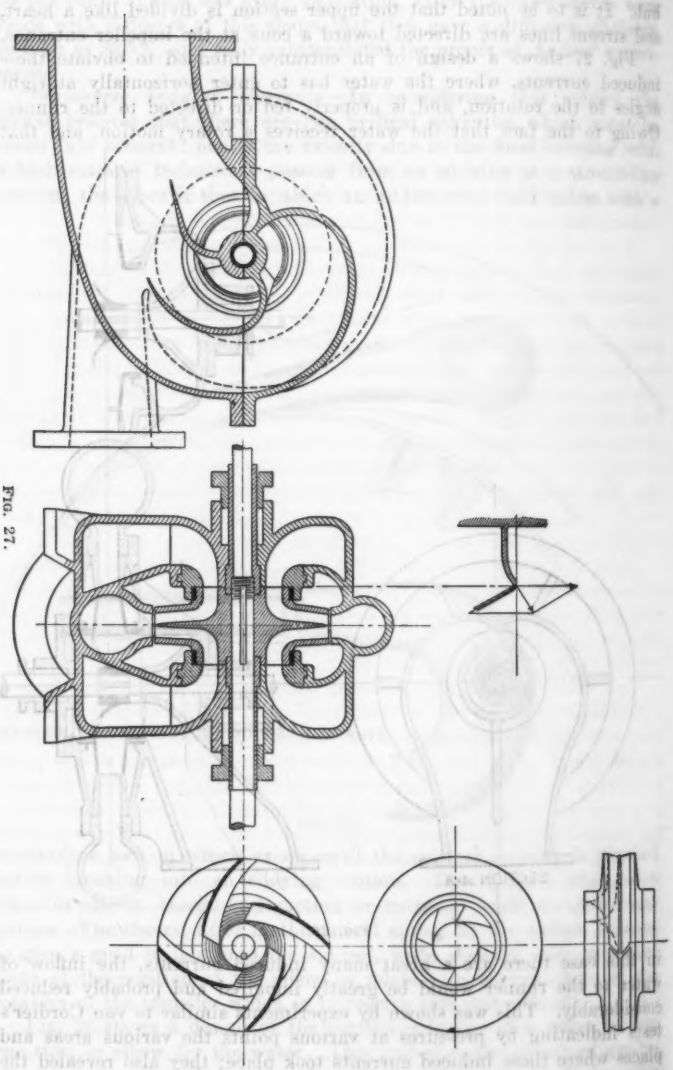


FIG. 27.

irregular manner and direct them into a well-defined path forming two or more streams or currents. Mr. de Laval.

The results showed that this theory was correct, and that, by making the suction chamber heart-shaped, shutting off the supply to the runners nearest the entrance, and dividing the stream into two or more nozzles, producing a volute chamber, the rotary motion of the

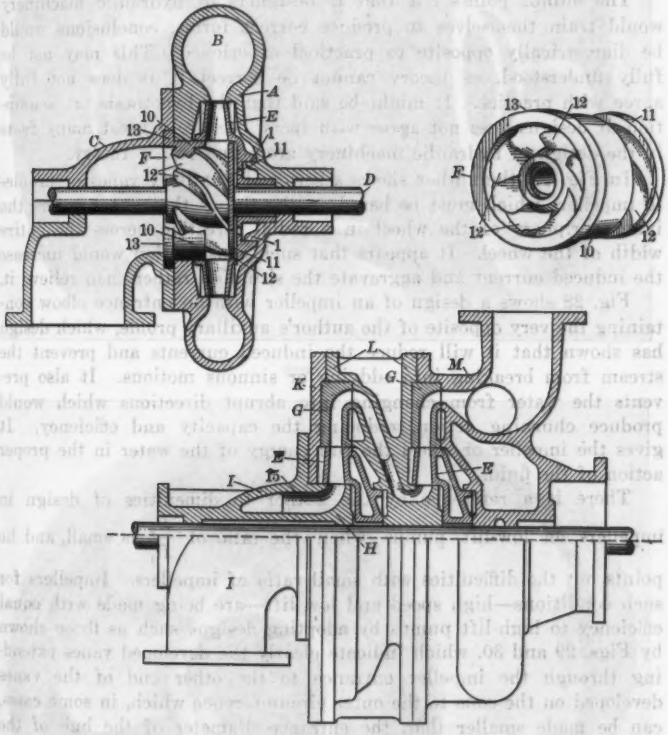


FIG. 28.

water, based on the law of rotation, swept the entrance channels clean, thus preventing a critical velocity which would have started or broken into eddying motion. The final results showed cases where the capacity was increased from 40 to 50% on otherwise the same construction.

In the conversion of kinetic into pressure energy, such as is to be considered in the design of hydraulic piping and machines, the passages have to be increased or decreased suddenly to reduce the

Mr. de Laval.

velocity of flow, and in converting a part of this into pressure energy many difficult problems are met, the laws of which are but little known. If, however, experiments are undertaken to establish these points, data are obtained which are suitable for all practical purposes, and these few results, given in a general way, may aid others to derive some information.

The author points out that if designers of hydraulic machinery would train themselves to produce correct forms, conclusions would be diametrically opposite to practical experience. This may not be fully understood, as theory cannot be correct if it does not fully agree with practice. It might be said that the hypothesis or assumption of designs does not agree with facts, because a great many facts in the design of hydraulic machinery are borne out by theory.

In Fig. 22, the author shows a series of auxiliary vanes or profiles to impellers which must be based on the theory that water enters the inner periphery of the wheel in a radial direction across the entire width of the wheel. It appears that such construction would increase the induced current and aggravate the situation rather than relieve it.

Fig. 28 shows a design of an impeller with an entrance elbow containing the very opposite of the author's auxiliary profile, which design has shown that it will reduce the induced currents and prevent the stream from breaking into eddying or sinuous motions. It also prevents the water from changing into abrupt directions which would produce churning action, reducing the capacity and efficiency. It gives the impeller or wheel the full energy of the water in the proper action of the fluid.

There is a reference by the author to difficulties of design in impellers on low-lift pumps when the ratio of $\frac{D_2}{D_1}$ is small, and he points out the difficulties with small ratio of impellers. Impellers for such conditions—high speed and low lift—are being made with equal efficiency to high-lift pumps by adopting designs such as those shown by Figs. 29 and 30, which indicate clearly the developed vanes extending through the impeller entrance to the other end of the vanes developed on the cone to the outer circumference which, in some cases, can be made smaller than the entrance diameter of the hub of the impeller. Such impellers give a remarkably high efficiency for low heads and high speed. The method of layout for such vanes is shown by Fig. 31.

Another method for inducing the flow to the impellers by guide nozzles is shown in Fig. 32. These references are given to show that there are other ways of obviating the defects referred to by the author, without resorting to expensive auxiliary partitions in the impellers.

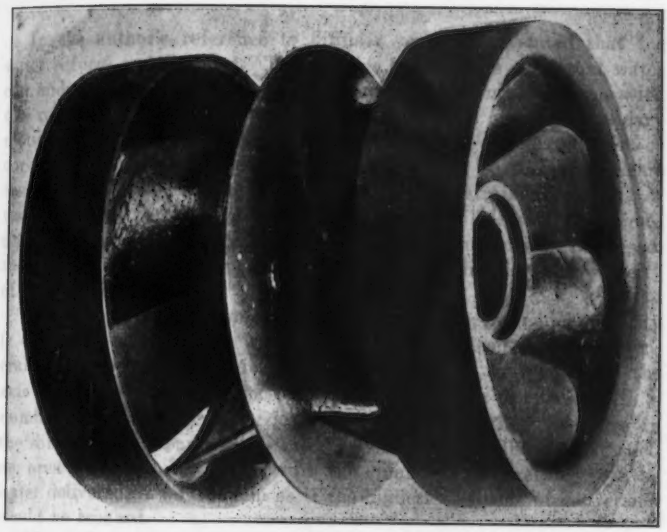


FIG. 29.

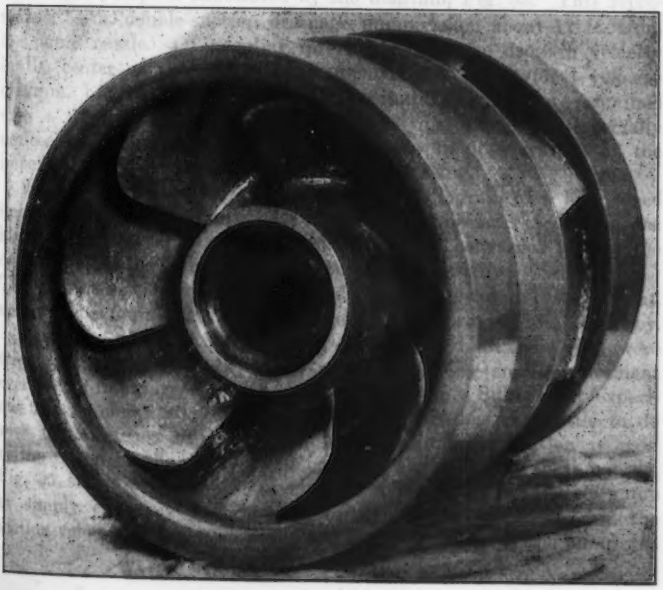
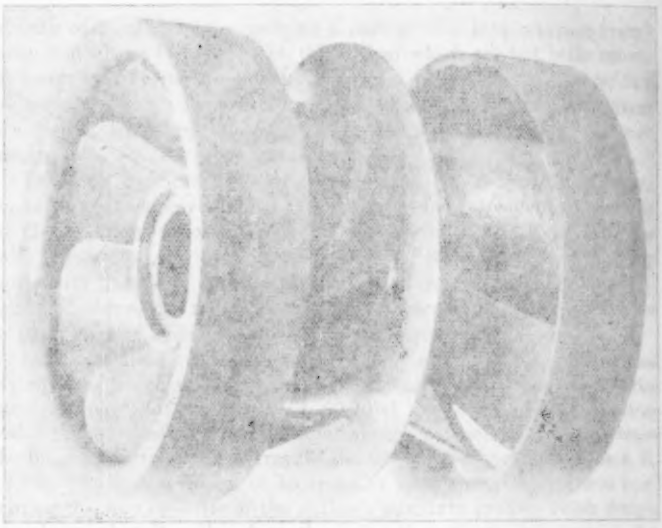
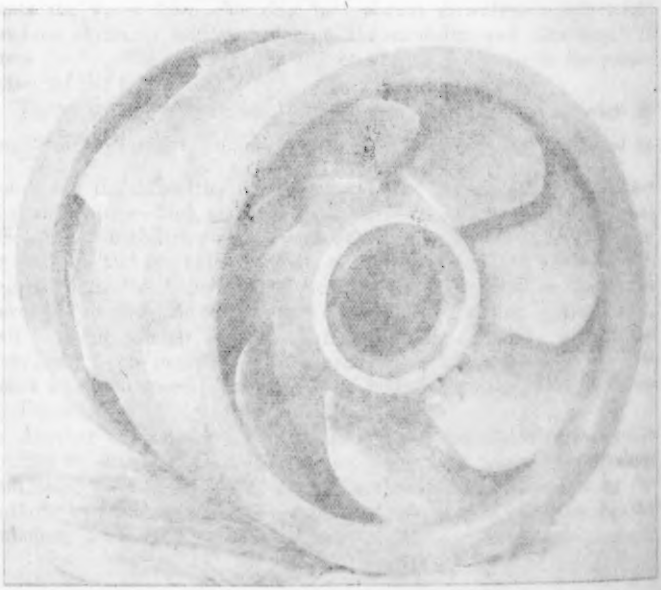


FIG. 30.





Mr. de Laval.

In the author's reference to diffusor vanes, it is stated that the water refuses to be guided. The writer's experiments show that water can and must be guided, but, at the same time, it must be done with due regard to the main motion of the stream lines and the secondary motion of the induced currents, introducing the necessary twist or corkscrew action with a nozzle discharge, which, so to speak, clears the passages. The writer believes that the unrestrained particles of water from the periphery of a rotating impeller will leave in a tangential direction at a tangential velocity. As the quantity is increased, the direction and velocity are modified by the direction and velocity of the radial flow.

The proportions of conical nozzles are known, in which velocity is converted into pressure with the least loss of head, and it is generally accepted that a conical pipe with a proper indirection to the axis is the best. As it is possible to surround the periphery with a number of such conical nozzles arranged so that their axes are in the direction of a particle of water leaving the impeller and of such an area at the throat as to suit the quantity and velocity of the water delivered, it would seem as if the highest efficiency of conversion would be reached.

The influence of runner curvature on the discharge velocity in centrifugal pumps is illustrated by the diagram, Fig. 33. This represents a 54-in. double-suction drainage pump, taken about 12 ft. from the pump nozzle. Of peculiar interest will be the drop in velocity in the center of the pipe when compared with the typical velocity diagram, Fig. 2, and with Fig. 22. Similar results were obtained with a number of different pumps. The disturbance is especially noticeable in high-speed pumps, due to the small curvature of the impeller as well as the small radius of the casing.

In Fig. 22, the author shows the velocity diagram of a high-speed impeller with radial components of respective pressures. Experience has shown, however, that the direction of flow converges toward the center, even if the ends of the curvature point radially. (See Fig. 34.) This fact must be kept in mind when designing the volute casing throat, and will be found of extreme influence in improving the efficiency.

Impellers with auxiliary profile, as suggested on page 892, were tried by the writer in 1910, but without showing any improvement over the ordinary type, and have been given up. Subsequent experiments have shown that the degree of curvature has very little to do with the efficiency of the pump, provided proper attention is paid to the shape of the inlet. The common suction elbow will not do. Fig. 35 shows a series of experiments made on a high-speed pump by simply changing the contours of the suction approach and the suction velocity. If it is considered that the efficiency of the pump was

Mr. de Laval.

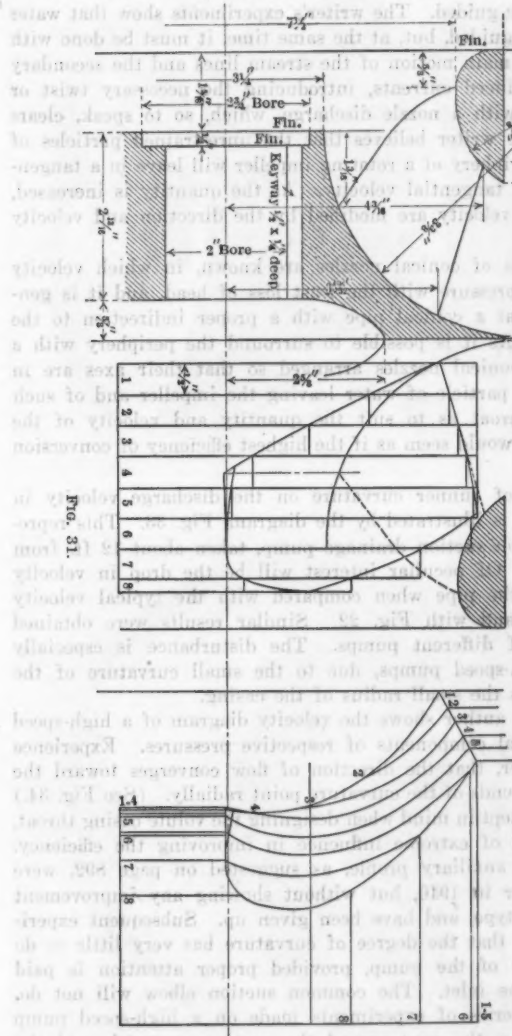


Fig. 31.

raised from 40 to 71%, without any change in the impeller or in the casing, the importance of this will be readily seen. The influence of a properly directed inlet is not so apparent on slow-speed pumps. This Mr. de Laval.

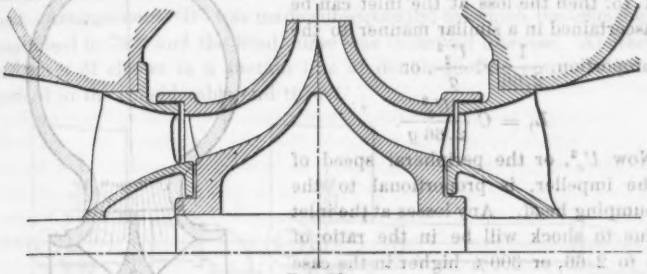


FIG. 32.

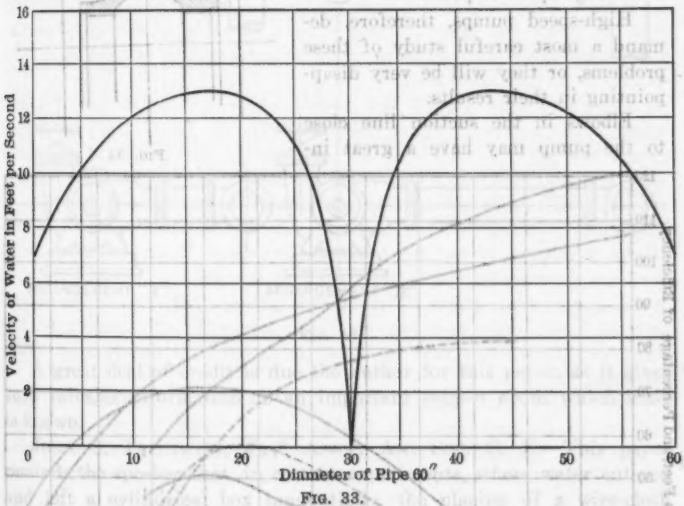


FIG. 33.

can be explained by the speed at which the water is picked up by the vanes. Take, for instance, an impeller with a ratio of

$$\frac{\text{outside diameter}}{\text{suction inlet diameter}} = 2,$$

then the velocity of the inlet, U_s, is equal to one-half the velocity at the outer diameter, U_2. Any shock losses at the inlet will be proportional to

$$\frac{U_s^2}{2g}, \text{ or } \frac{1}{8}\frac{U_2^2}{g} L = C\frac{U_2^2}{8g}.$$

Mr. de Laval.

Let us take now an extreme high-speed impeller with a ratio of $\frac{D\,a}{D\,s} = 1.15$: then the loss at the inlet can be ascertained in a similar manner to the proportion, $\frac{1}{2.66} \times \frac{U_2^2}{g}$, or

$$L_1 = C\,\frac{U_2^2}{2.66\,g}.$$

Now U_2^2, or the peripheral speed of the impeller, is proportional to the pumping head. Any losses at the inlet due to shock will be in the ratio of 8 to 2.66, or 300% higher in the case of the high-speed impeller.

High-speed pumps, therefore, demand a most careful study of these problems, or they will be very disappointing in their results.

Elbows in the suction line close to the pump may have a great in-

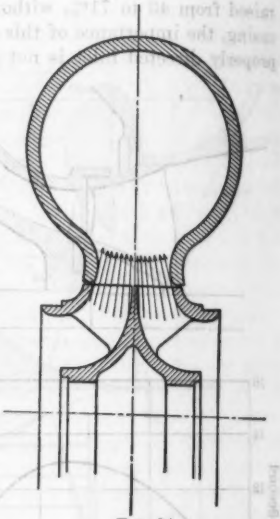


FIG. 34.

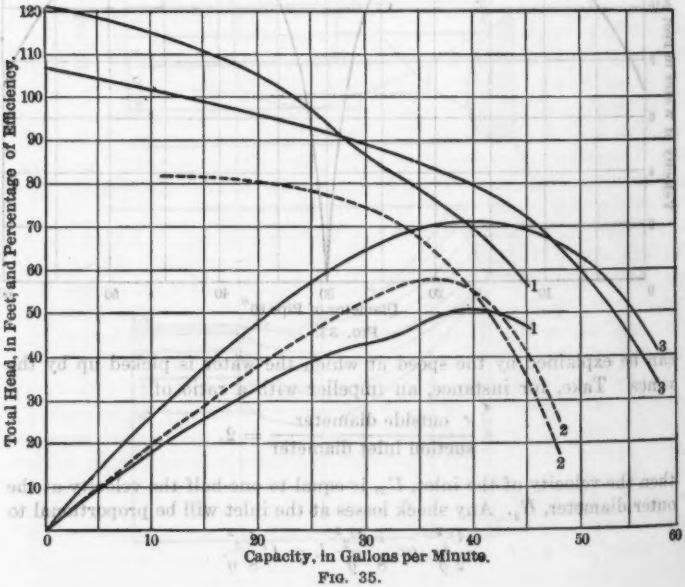


FIG. 35.

fluence on the characteristics of the latter. Fig. 36 shows a 14-in. pump. Arrangement "A" was made with a standard long-sweep elbow, 3 ft. from the pump nozzle, with a pump efficiency of 65 per cent. Arrangement "B" was made subsequently, by which the efficiency was raised to 75% and the head curve was improved likewise. Another bad effect of elbows in a suction line of double-suction pumps will be noticed in the considerable end thrust. Mr. de Laval.

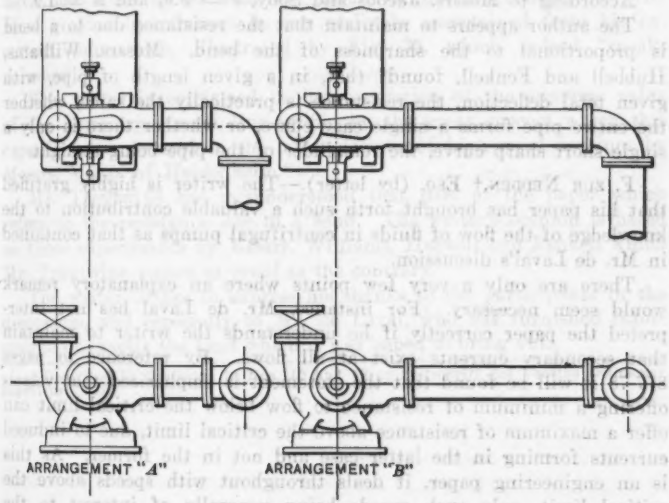


FIG. 36.

A great deal of credit is due the author for this paper, as it gives such valuable information on an important subject about which little is known.

JOHN C. TRAUTWINE, JR.,* ASSOC. AM. SOC. C. E.—This paper reminds the speaker that, in certain experiments, where water entered and left a cylindrical box tangentially, the placing of a wire-cloth screen in the box, instead of increasing the loss of head, as expected, diminished that loss, evidently by diminishing or destroying the centrifugal force, due to the previously unchecked rotary motion of the water. Mr. Trautwine.

The speaker is reminded also that Messrs. Gaskell S. Jacobs and F. A. Sooy have described† a simple method, devised by Mr. Buckner Speed, of measuring the velocity of water flowing through a pipe in which a bend occurs. This consists simply in tapping the bend on

* Philadelphia, Pa.

† *Journal of Electricity, Power and Gas*, San Francisco, July 22d, 1911.

Mr. Trautwine.

its outer and inner sides. Then, if h equals the difference in pressure, in feet, between the outside and the inside of the bend, R equals the radius, in feet, of the bend described by the pipe axis, and d equals the pipe bore, in feet; we have, for the velocity, in feet per second,

$$v = c\sqrt[n]{h\frac{R}{d}}.$$

According to Messrs. Jacobs and Sooy, $c = 5.6$, and $n = 1.9$.

The author appears to maintain that the resistance due to a bend is proportional to the sharpness of the bend. Messrs. Williams, Hubbell and Fenkell, found* that, in a given length of pipe, with given total deflection, the resistance is practically the same, whether the entire pipe forms a single easy curve, or whether there is only a single short sharp curve, the remainder of the pipe being straight.

Mr. zur Nedden.

F. ZUR NEDDEN,† ESQ. (by letter).—The writer is highly gratified that his paper has brought forth such a valuable contribution to the knowledge of the flow of fluids in centrifugal pumps as that contained in Mr. de Laval's discussion.

There are only a very few points where an explanatory remark would seem necessary. For instance, Mr. de Laval has not interpreted the paper correctly if he understands the writer to maintain that secondary currents exist at all flows. By reference to pages 877-78 it will be found that the statement is emphasized that vessels offering a minimum of resistance to flow below the critical limit can offer a maximum of resistance above the critical limit, due to induced currents forming in the latter case and not in the former. As this is an engineering paper, it deals throughout with speeds above the critical limit, only such speeds being generally of interest to the engineer; those below that limit fall mostly within the domain of the physicist.

The two critical speeds which Mr. de Laval mentions on page 900 are of no importance to the engineer. From the use he makes of the term "critical velocity" later (on page 903), it would seem that he means by it something quite different from the well-defined scientific term.

It certainly is a source of great satisfaction that the results obtained from diffusors such as Fig. 25 confirm the writer's theories in such a remarkable way.

Fig. 26 is an example of the practical application of the writer's recommendation to steady the flow at the entrance to impellers by decreasing the area. The same is true of the auxiliary inlet vanes such as Fig. 32.

* *Transactions*, Am. Soc. C. E., Vol. XLVII, April, 1902.

† New York City.

Mr. zur Nedden.

It is gratifying to have as complete a vindication of the writer's thesis as the following sentence on page 907 by Mr. de Laval, one of the highest authorities among pump manufacturers in the United States: "The common suction elbow will not do."

The alleged bad experiences of Mr. de Laval with his auxiliary profiles is not considered as conclusive evidence against the expedient recommended by the writer. The shape of the entrance chambers might have had much to do with the failure of his experiments in 1910. How great this influence is, can easily be ascertained from his excellent comparison, illustrated by Fig. 35, which indeed "speaks volumes".

The writer is confirmed in his conviction of the supreme value of the auxiliary stream line profile by the great success which this expedient has secured in the latest designs of high-speed turbines by Messrs. Voith, of Heidenheim.*

Mr. Trautwine has misunderstood that part of the paper which refers to the resistance due to a bend. The writer's opinion is based on those experiments by Messrs. Williams, Hubbell and Fenkell which Mr. Trautwine quotes as proof to the contrary.

The writer wishes to express his thanks to the participants in the discussion, and especially to Mr. de Laval, who has furnished information of exactly the kind which it was hoped to bring out.

* Described by Oesterlen in Nos. 40 and 42 of *Zeitschrift* des Vereins Deutscher Ingenieure, 1915.

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Paper No. 1365

SECURE SUBWAY SUPPORTS*

By A. B. Lueder, M. Am. Soc. C. E., and W. J. R. Wilson, Esq.

With Discussion by Messrs. H. G. Moulton, Richard A. Fiesel, J. H. O'Brien, B. C. Collier, Lazarus White, and A. B. Lueder and W. J. R. Wilson.

Synopsis

The object of this paper is to describe the problems met in the construction of Section 13 of the Lexington Avenue Subway, New York City, to state how those problems were solved, and to give the reasons for the methods selected.

Deep and wide rock cuts made Section 13 one of the most difficult parts of the subway work. It was necessary to have an improved system of street supports in order to insure safety, and attempts were made to forsee such dangers as those caused by blasting and by rock slides. The advantages to the contractor in using massive supports and continuous steel beams are argued, and descriptions of the towers and beams are given, as well as the methods of putting them in place.

The main conclusion of the paper is that the safety of city streets during subway construction will be better guaranteed if a comprehensive design of street supports, made by competent engineers, is bid on as part of each contract.

This paper describes a street supporting system, designed by the writers and used in the construction of Section 13 of the Lexington

* Presented at the meeting of February 16th, 1916.

Avenue Subway, New York City, for the contractors, McMullen-Snare and Triest, during the period, 1912 to 1914. Section 13, which extends from 118th to 129th Street, was one of the most difficult on the New York subways, owing to the varying depths—from 30 to 60 ft.—and the varying widths—from curb to curb in the shallow parts to practically the full width of street from building to building in the deep section. The corkscrew arrangement of the tracks in this section made the design of the bottom most irregular, often requiring four different levels, with a variation as great as 20 ft. in the rock bottom across the cut. The permanent structure was so irregular that practically every one of the six hundred steel bents was different. The material excavated varied from quicksand to the hardest kind of rock. The rock surface was also very irregular, being sometimes at the street surface and sometimes not found 60 ft. below. The rock itself was typical Manhattan schist and Inwood limestone, probably harder on the whole than that found in the lower part of the city, and containing many mica seams and other faults so common in New York rock. Very sharp dips transversely across the cut were frequently encountered.

In the design of the street supporting system, it was felt that the most important feature was the likelihood of New York rock to slide suddenly—a very serious danger in a deep cut. It was decided, therefore, that the supports should not rest on the rock at the sides, but on the rock bottom in the center of the cut, where they would be farthest from danger if a slide occurred, and would be directly under the street-car tracks, the heaviest load to be carried.

Any system of street support, to be effective from the standpoint of the contractor's engineers, must guarantee absolute safety to the public, must give ample working room, and must be economical as regards cost of construction and erection. In the special case, adaptability to the conditions required a most flexible scheme. A contractor cannot afford to pay damages to the public, nor does he wish that kind of advertising. The public is entitled to a design with a large factor of safety, because its members must of necessity use the streets, and are helpless to safeguard themselves against unseen dangers beneath the street surface. The contractor may feel that he can risk his money or his plant sometimes on work, but the contractor's engineer should not involve the public in any risk on designs that he makes. A subway under construction always causes

a certain amount of inconvenience to the public, especially to the business men along the route, and has never been entirely free from a certain number of minor accidents, due to concussion from blasting or irregularities in the temporary street surface. The life of wooden decking is short. The public will be glad to have the job completed in the shortest possible time.

The contractor's profits will vary to a large extent with the time he takes to do the work. Every contractor realizes that there are twice as many Sundays, nights, holidays, and bad-weather days in two years as there are in one, and these days mean expenditure without estimate. The money he pays out for watchmen and rentals, and a large part of his overhead expense, will vary according to the time required to do the work. Contracting is a peculiar business in that the profits are distinctly limited by competition, but there is no limit to the losses. The public and the contractor are agreed that subway work should be done with all speed. This is only possible with the use of the most modern plant, requiring that under the street, as above, the contractor must use hoisting engines, heavy skips and cars, and eliminate all the hand labor possible. This necessitates plenty of working room, clear of street supports. Moreover, the fewer the obstructions, the better will be the light, the greater the safety to the workmen and the greater the security because of ease of inspection and adjustment of the street supports. Instead of filling the cut with posts, each supporting an unknown load, with many wedges always uncertainly tight, there should be provided systematically-placed large supports, carrying known loads, by their size and construction better suited to take blasting shocks or to resist slides, and constantly in plain sight. Any impending danger or weakness will then immediately show itself, and, being accessible, can be easily remedied with the use of heavy machinery. In the construction of the permanent structure, for the placing of steel and concrete, the advantages of working room free from obstructions means fewer accidents, more profits, and better permanent work.

The designer, having in mind a system amply strong to carry the street loads, with supports as far removed as possible from the effects of slides, must consider how it will meet the danger due to blasting. The exact effect of a blast cannot be foretold by any one, because of the constantly changing nature of the rock which is

FIG. 1.—TYPICAL ROCK SLIDE, IN LEXINGTON AVENUE.

FIG. 2.—TYPICAL ROCK SLIDE, IN LEXINGTON AVENUE.

being blasted, but experience should teach the designer to make the parts of his system exposed to blasts amply strong to withstand the maximum effect of shocks from flying rock—with intelligent blasting. Rather than strengthen the system still further to withstand careless or excessively heavy blasting, the contractor should have the blasting done by intelligent, experienced men. In a subway cut, this means taking into consideration three points: First, as always, safety, then maximum output, and then minimum of cost for labor and powder used. The blaster should be able to place and load the holes so that the rock will be thrown clear of supports against the sides of the cut, or into a clear space. The intelligent blaster will study the rock constantly, will understand where to drill the holes, and will be thoroughly conversant with the use of dynamite. It is desirable to blast as seldom as possible; that means getting a maximum quantity of muck with each blast. The nearer the blasting and excavation approach the open-quarry method, the greater will be the output, the less the powder used, and the less the number of small flying stones —all this tending to greater safety and lower cost.

The blaster, however, will be helpless if he has not a clear space, free from supports, into which he can throw large quantities of rock in safety. Such blasting brings out the rock in large pieces, and to prevent numerous interruptions to everybody from small pot-hole blasts to break them up, means must be provided for taking heavy masses of stone from the face to the dump. It is the duty of the designer to give a blaster this leeway, and to provide room and strength in his supports, so that the muck foreman may use heavy machinery to handle these large stones. On Section 13 locomotive cranes and derricks were used successfully at the face, and the supports provided stiffness and strength necessary to load heavy masses at the face by using hoisting engines and lines rigged through blocks fastened to the beam supports overhead.

Even assuming all carefulness on the part of the blaster, a designer will still have to meet, in the deeper cuts, the danger of slides in the face after a blast. This danger is small in shallow cuts, but is a most serious menace in a cut 60 ft. deep. In the side-walls small local slides may be prevented by steel pins or by shoring from the bottom with timber or concrete. Provision against large side slides in deep cuts, in the opinion of the writers, should be made in the

design of the permanent work furnished the contractors. It is a practical impossibility, owing to the numerous fissures and mica seams, to carry along a deep vertical face in New York rock. Careful inspection of the rock at the face, and search for cracks and seams on the top of the rock ahead, will sometimes give warning, but will not take care of a slide when it finally occurs. To meet this danger, the design on Section 13 provided for a span over the face long enough to allow of a support at the toe and one back of the top of the slope formed by a slide. Supports resting on the working face itself were too dangerous and too much in the way to be considered. In the deep cuts the width was so great that a transverse support spanning the cut and resting on the rock at the sides would have been expensive, and would have been useful at one point only, and, owing to the chance of slides, was considered too dangerous. In shallow and narrow cuts a transverse support, of ample strength to carry the street, and of a massive type to resist blasting shocks, was used, and rested well back on the rock at the sides. Sometimes, where the rock was uncertain, an extra support, consisting of a massive built-up post, was placed where it would cause least interference in blasting and excavation. Their size and mass (usually 24 by 36 in.) made these posts safe from chance shocks.

From the standpoint of the man who is actually building the subway, it is usually impracticable to keep the erection of the permanent structure close up to the excavation. The excavation being the largest part of the work of subway building, fast progress necessitates that the system of work shall in no way retard it. If the permanent work follows closely behind the face in a rock cut, it will interfere seriously with excavation, and the excavation will interfere seriously with the permanent work—for instance, interruptions due to blasting. The difference in elevation of the concrete floor and the excavated cut would hinder excavation, and the muck passing through the permanent work would hinder the steel workers, the carpenters, and the concrete men. Thus, the labor cost of both classes of work would increase, and, the progress being slower, the overhead cost would also increase. It has frequently happened in subway construction that the contractor, owing to changes or delays in making the working drawings, has been unable to obtain the materials for the permanent work when needed, and has been forced to keep the excavation

FIG. 3.—A HEAD ROCK RUNNING ACROSS THE CUT.

FIG. 4.—ONE SIDE OF A ROCK CUT, WITH TOWERS IN THE CENTER.

FIG. 5.—A LONG-FACE SPAN, SHOWING TOWERS, CONTINUOUS BEAMS, AND STEEL NEEDLES.

FIG. 6.—WORKING FACE UNDER LONG SPAN WITH NO POSTS.

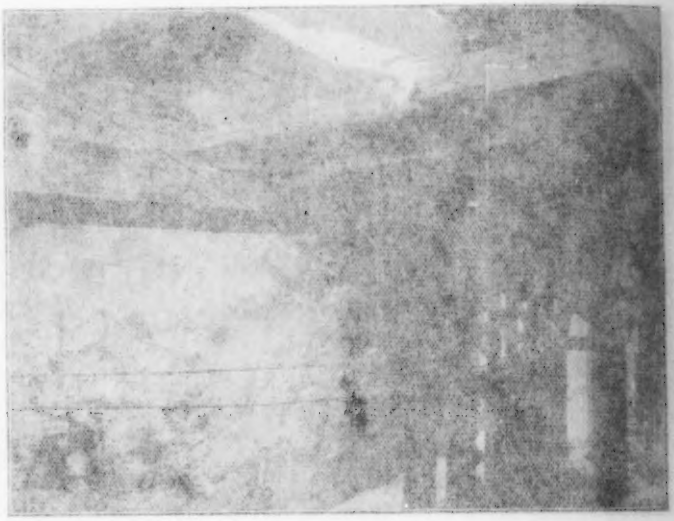
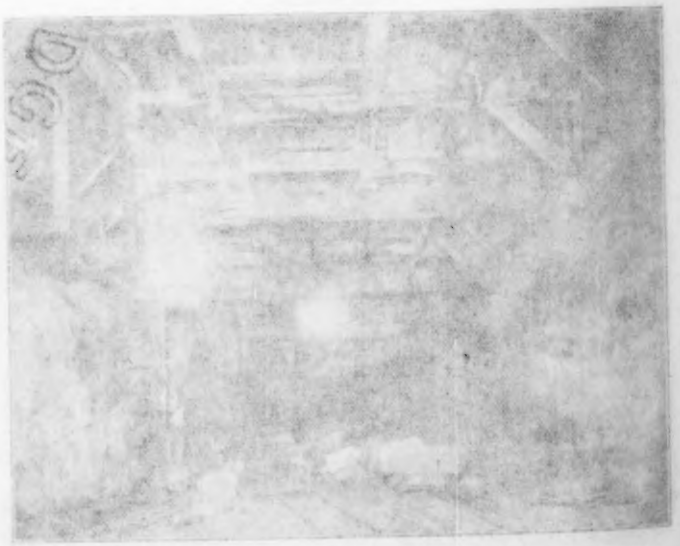


FIG. 7.—BLASTING FOR DEEPER LEVEL: ILLUSTRATING THE FACT THAT, WITH AMPLE CLEARANCE, THERE IS ABSENCE OF DANGER.

FIG. 8.—METHOD OF TRANSVERSE SUPPORTS UNDER CONTINUOUS BEAMS IN NARROW, SHALLOW CUT.



open at points where he might have placed the permanent work. With the system adopted in Section 13, using steel supports, there could be no deterioration in the main parts to impair materially the safety of the structure.

To carry a 50-ft. length of street on a single span indicates the use of steel. If the designer provides for this span over the face, with good supports at each end, he may have a large factor of safety until one of the supports is shot out, and then he will be left with no factor of safety, and a considerable length of street may drop at once. Complete safety here requires him to have still a factor of safety in reserve by the provision of other supports back of the first ones, so that the breaking of one of these will still leave the structure safe, though that factor will be smaller, owing to the increased span. If bents are used for supports, they should be in pairs, so that one may always be in reserve; and the two bents should be thoroughly braced and tied, forming a tower with a broad base, which cannot be easily overturned. The use of two independent bents, each capable of carrying the load, gives a double factor of safety, and permits of the repairing or shifting of either bent.

To get space and clearance in the cut by combining numerous separate supports into one large one necessitates the use of long-span steel supports between towers. Short spans would not give flexibility enough for the shifting of supports. This point was very important on Section 13, because deeper levels had to be blasted out after the main excavation had been completed, and these extra cuts, sometimes 20 ft. deeper, frequently came directly under the supports. In placing concrete in floors, the extra factor of safety from the use of double bents permitted the contractor to sling to the beams above and then cut off one of the bents, and concrete to the middle of the tower. Then the short bent was wedged on the concrete, and the other bent was cut off, and the concrete placed, thus rendering it unnecessary to leave any posts or holes in the concrete. All needles were placed 10 ft. apart, so that they came exactly between two bents of the permanent steel. A 40-ft. beam was the most convenient length to use, and, as the tower bents came under the needles 10 ft. apart, this left a clear span of 30 ft., the full width of the street, to be carried by the beams with a factor of safety sufficient to allow a 40-ft. span when necessary.

If the beams had rested on bents with broken joints, the towers could not have been shifted, and this and the resulting extra strength led to the use of continuous beams. The beams were used in pairs, spliced at the joint to make a box girder, with long plates on top and bottom flanges and a wooden filler between the webs. To get the extra strength for the span needed over the long working face (55 ft. in the case of Section 13), extra pairs of beams, 100 ft. or more in length, were used on each side of the first pairs and pulled along as the face progressed. The continuous beams were carried in the center of the street, slightly outside the loads from the car tracks. It was necessary to transfer the loads from the driveway and sidewalk to these beams. The main part of the load from the driveway was carried on steel needles the ends of which were cantilevered over from the continuous beams on which they rested.

The load from the sidewalk, which, of course, was much less, was carried on wooden extensions of the steel needles. These extensions were fitted against the web and bolted securely to the steel needle with a long splice, making a joint good for bending, or for compression where it acted as a brace. For facility in posting, and to prevent the needle from rolling, it was packed on both sides with timber so that the whole needle had the shape of a square timber. These wooden plates helped in the placing of the needle by allowing easy fitting at the ends, and also served a valuable purpose, when driving a top lift in rock, in protecting the steel from flying rock while blasting. The needles acted as transverse braces across the cut to keep the whole system rigid, and were wedged against the rock or against the foundation walls of buildings. There were also provided raker struts, from the needles to niches in the rock, which helped to stiffen the structure and minimize vibration. To prevent the continuous beams from rolling, wooden bracing, with heavy turnbuckles on top and bottom of the beams, were put in every 20 ft. between each adjoining pair; similar bracing and buckles were used between adjacent needles.

On Section 13 it was found advantageous to take out a 16-ft. first lift, giving plenty of room for loading muck with machinery. A steel needle has a greater margin of safety than a wooden one at the face, and requires fewer posts. The continuous beams were spliced, and were kept a short distance back as the face advanced, permitting

FIG. 9.—TOWER BENT, BROKEN OFF BY BLAST, SUSPENDED TO BEAMS ABOVE, WHICH ARE CARRYING SAFELY THE LOAD OVER THE EXTRA SPAN BACK TO THE NEXT SUPPORT.

FIG. 10.—TOWER BENT, BROKEN OFF BY BLAST, LASHED TO SUPPORTING BEAMS OVER THE WORKING FACE WHILE BEING REPAIRED.

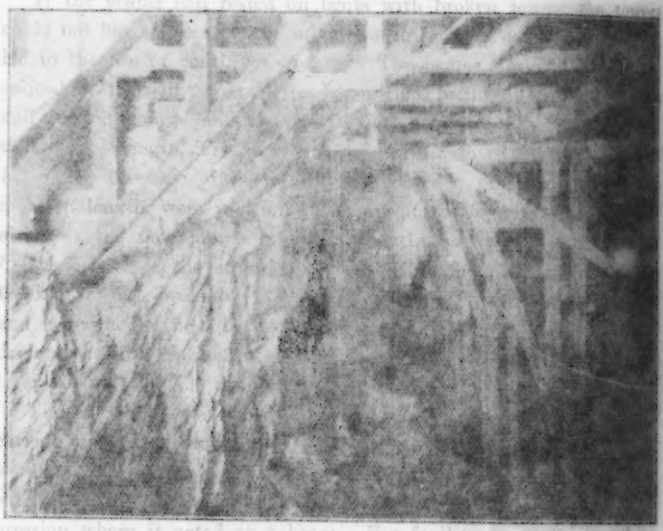


FIG. 9.—TOWER BENT, BROKEN OFF BY BLAST, SUPPORTED TO BEAMS ABOVE WHICH ARE CARRYING SUPPLY THE LOAD OVER THE EXTRA SPAN BACK TO THE NEXT SUPPORT.

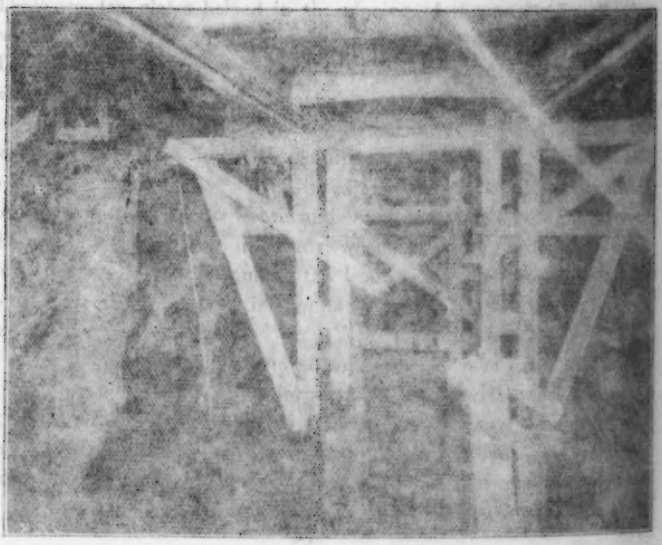


FIG. 10.—TOWER BENT, BROKEN OFF BY BLAST, LEANING TO SUPPORTING BEAMS OVER THE WORKING PLACE WHILE BEING REPAIRED.

FIG. 11.—TOWERS IN DEEP CUT, LOOKING DIAGONALLY ACROSS.

FIG. 12.—TOWER SUPPORTED WHILE EXCAVATION FOR DEEPER LEVEL IS BEING MADE UNDER IT.

FIG. 11.—TOWERS IN DEEP CUT, LOOKING DIAGONALLY ACROSS.

FIG. 12.—TOWER SUPPORTED WHILE EXCAVATION FOR DEEPER LEVEL IS BEING MADE UNDER IT.

the use of still fewer posts, thus allowing the placing of tracks and cross-overs where they would most facilitate fast excavation.

As the towers were to carry heavy weights, they had to be well made, and to avoid depending on nails, they were accurately framed, bored, and match-marked in the contractor's yard by a competent foreman, working from a blue-print, and were carefully inspected before being sent to the cut, where the erecting gang put them together on the ground in the bottom of the cut and erected each as a unit. This made a great saving in time and lumber. The employment of high-grade timbermen in the yard and high-grade erectors in the cut specialized the work, and made it economical. The use of a long span over the face permitted a leeway of 2 or 3 days, and the choice of the most favorable opportunity during that time for the erection of the tower. The erecting was usually done at night, in order to prevent any interruption of the excavation during the day, and to permit the erecting gang to work continuously, without stopping for blasts; this also allowed the use of the heavy machinery needed in the day by the excavating gang. It was an important consideration for the contractor to have all possible lumber work done in his yard where he had at hand power tools, such as cranes for lifting heavy sticks, power saws, air augurs, etc. Left-over pieces of timber were valuable for other purposes, but in the cut they were a nuisance, and were wasted. In the yard the foreman had the blacksmith shop near him, the men had places for their tools, and they worked in daylight, without troubling about blasts or workmen over their heads.

As before stated, each tower consisted of two bents, as shown by Fig. 13, and these bents and the bracing between them were all fastened with $\frac{7}{8}$-in. bolts and dock washers. The use of bolts led to simplicity in construction and dismantling, prevented the spoiling of lumber by nails, and left nothing to the discretion of the workmen when assembling the bents. The use of $\frac{7}{8}$-in. bolts, dock washers, and a uniform width of 12 in. for lumber standardized the work, and such standards were maintained throughout the design.

Some of the advantages due to special features of the design described were peculiar to Section 13, but the writers believe that the principle of continuous beams under the car tracks, supporting steel needles carrying the street loads, is necessary in most of the subway rock cuts for safety. The principle offers great advantages

in earth cuts also, because it allows keeping the heavy vertical loads off the cross-bracing, and serves to support the cross-bracing, thus replacing the transverse beams often used across an ordinary earth cut. A great advantage that the continuous beams have here is that they permit of shifting or repairing the cross-bracing without the removal of posts between sets of bracing.

One possible danger which this or any other system of street support and temporary shoring does not, and cannot, cover is the danger to buildings and their occupants along a deep rock cut from a wholesale side-slide where the cut is only about a foot from the building line. Where the danger of a local slide has shown itself in cracks or seams, it has been customary to take care of that particular spot by immediate underpinning, or by putting the building on cantilevered beams (thus taking the weight of the front wall off the edge of the cut), or by shoring of different kinds. There are cases, however, where the rock may be bad and there may be no visible evidence of the fact, as the top of the rock under the building cannot be examined. Up to the present time no general provision for safety from this danger has been made, but, in view of the fact that such a slide may take place, and of the probable large loss of life it might cause, it would seem advisable to provide for the danger, even during the time of construction, by the permanent design. This can be done, without excessive cost, by a perfectly feasible scheme. From the contractor's standpoint in bidding to-day on subways, the smaller the factor of safety the bidder intends to use in his street supporting system, the better his chances of securing the contract.

It would be conducive to greater safety if the main features of the system of support to be used were designed before letting, and bid upon as part of the contract. Systems have been designed piecemeal from the bottom up by combining the ideas of timbermen and other workmen, but the writers believe that the safety of city streets would be better guaranteed with a comprehensive design made by competent engineers. The authority and responsibility connected with the design of street supports, and the inspection of them when placed, or when shifted, or removed, should be in as few hands as possible.

The inspection of the street supports and the rock faces is of the utmost importance in subway excavation. The system which permits

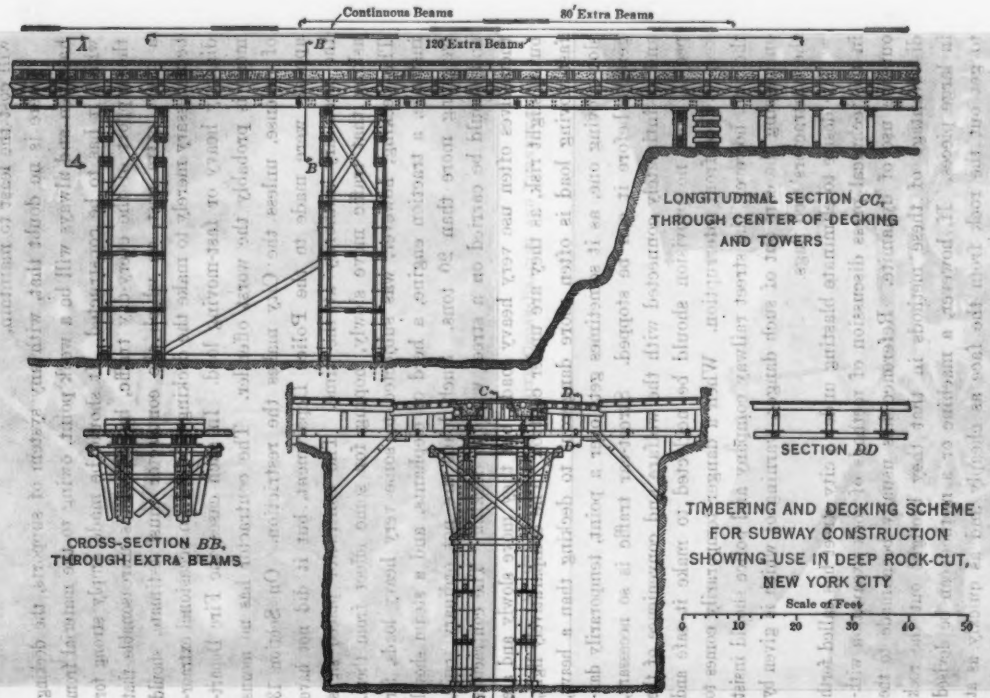

Continuous Beams
80' Extra Beams
120' Extra Beams
A
B
C
D
LONGITUDINAL SECTION CC, THROUGH CENTER OF DECKING AND TOWERS
SECTION BD
CROSS-SECTION BB, THROUGH EXTRA BEAMS
CROSS-SECTION AA
TIMBERING AND DECKING SCHEME FOR SUBWAY CONSTRUCTION SHOWING USE IN DEEP ROCK-CUT, NEW YORK CITY
Scale of Feet
0 10 20 30 40 50

FIG. 13.

of the most rapid and thorough inspection, with the minimum of necessity for any inspection, will be the least subject to danger, and will cost the least to maintain.

There is no doubt that, with any system of supports, the decking has been and always will be a weak point, owing to the material from which it has to be constructed. It should be made amply strong for the support of the every-day traffic, but it seems unreasonable that a great extra cost, on which the contractor must estimate, should be necessary merely to make the decking safe for an occasional extraordinarily heavy or fast-moving load. In such cases the Fire Department is probably the worst offender. The contractor has no means of defense, unless the City makes the restriction. On Section 13 appeals were made to the Police Department, but it did not have the necessary authority at the time. The contractor placed signs asking that traffic move slowly, hoping for some effect from them. The decking, however, was subjected to some very heavy loads, for instance, a traction engine, a herd of elephants, and a steam shovel weighing more than 20 tons. Such loads are not ordinary traffic, and should be carried on a street without decking. The contractors themselves often use very heavy loads, but they move slowly and are but a slight risk, as they are under control. The comparatively light, fast-moving load is often more dangerous to decking than a heavy, slow-moving one, as it sometimes gets over a point, temporarily dangerous, before it can be stopped. Street-car traffic is so necessary, and so intimately connected with the welfare and convenience of the people, that no provision should be neglected to make it safe and keep it free from interruption. When a danger temporarily comes to the line, however, the street railway company and police should insist on keeping the cars out of such danger, warning of which is given by the contractors' red flags.

The desire to eliminate blasting under a city street has called forth in the technical press discussion of methods of rock excavation without the use of dynamite. Reference has usually been made to the disadvantage of these methods in that they brought out the rock in large pieces. If, however, a machine or a method can be devised to get out the rock from the face as cheaply and as quickly as at present with dynamite, the large-sized pieces would be no objection to its use with a supporting system such as adopted on Section 13.

FIG. 14.—CONTINUOUS BEAMS, WITH CROSS-BRACING AND TURNBUCKLES.

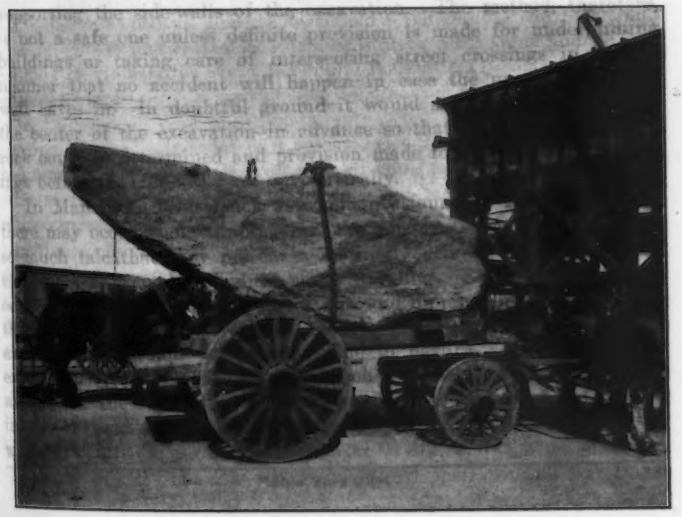
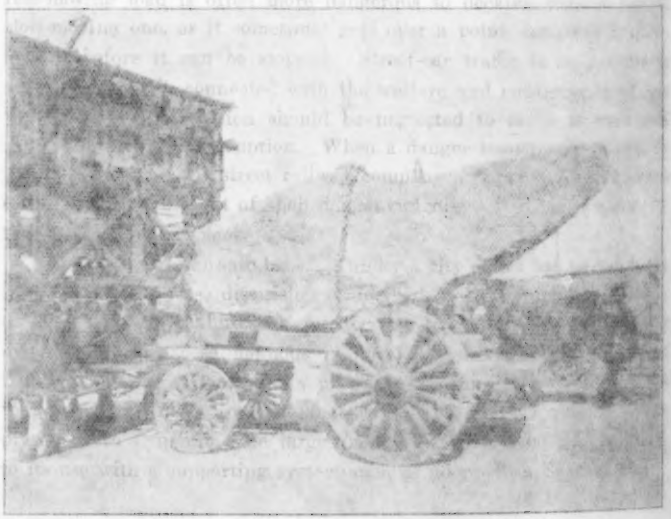


FIG. 15.—25-TON ROCK, HANDLED IN ORDINARY COURSE OF THE DAY'S WORK.



DISCUSSION

Mr. Moulton.

H. G. MOULTON,* ESQ. (by letter).—The method of shoring and street support discussed in this paper is excellent, and fully entitled to classification under the heading "Secure Subway Supports".

The essential feature of safety in this method does not lie in the massive cantilever tower supports in the center, but in the continuous steel longitudinal beams supported thereon. The same ends of safety may be attained—and it is probably open to question as to whether they may or may not be attained at an equal or lower cost—by supporting the continuous beams on transverse girders resting on posts which are in turn securely braced so that they act as retaining timbers for the vertical rock walls of the cut.

Any method of shoring which does not remove the possibility of a sudden local collapse in the side-walls of the excavation is dependent for its safety on continuity of the decking support, so that the decking will carry over if a local rock-fall destroys any one essential supporting member. The same criticism is applicable either to transverse box girders resting on posts adjacent to the walls, or to center supporting towers. In each case any particular support may be destroyed by blasting or by caving of the side of the excavation, and in such an event the entire load of two panels must be transferred immediately, with a fair margin of safety, to adjacent supports.

The method of shoring discussed herein makes no provision for supporting the side-walls of the excavation. The method, therefore, is not a safe one unless definite provision is made for underpinning buildings or taking care of intersecting street crossings in such a manner that no accident will happen in case the unsupported rock wall caves in. In doubtful ground it would seem necessary to open the center of the excavation in advance so that the character of the rock could be determined and provision made for underpinning buildings before the side-walls are cut vertically.

In Manhattan schist such as commonly found in New York City, there may occur seams dipping into the cut at a steep angle, carrying so much talc that they present a distinctly defined plane with practically no frictional resistance against sliding. Should such a seam occur, with a strike parallel to the cut, and dipping in such a manner that it would be cut across or nearly exposed at the bottom of the excavation, it would be quite possible to have a very serious slide extending over a great distance. Such a slide might bring down a number of the supporting towers placed in the center of the cut, as in the method now under discussion, and thereby bring down the whole system.

* New York City.

Mr. Moulton.

In regard to the remarks made by the authors in reference to excessive loads on the decking—that in such cases the Fire Department is probably the worst offender—it has been the writer's general observation that the contractor himself is apt to be the worst offender. With the writer, it has not been an uncommon occurrence to stand at the side of one of the warning signs prohibiting 8-ton trucks from passing, and at the same time observe a contractor's truck, weighing with its load nearer 15 tons than 8 tons, go by at the maximum speed of which it is capable.

The writer takes very decided exception to the authors' main conclusion—that the safety of city streets during subway construction will be better guaranteed if a comprehensive design of the street supports, made by competent engineers, is bid on as a part of each contract. The shoring system in subway excavation is an essential part of the method of construction, and the contractor must devise that system to fit with his usual methods of work and his equipment. It is entirely possible to handle subway excavation safely in a great many different ways. The contractor may use smaller timbers and plenty of them, with good detail, working in small units and keeping his steel structure close to the face. Or he may use large units, so firmly braced that they can stand the shocks of blasting or caves. In this method very great attention must be paid to detail of connections. Or he may provide a continuous decking support which will carry over in the event that he loses any particular point of support. Different contractors have gangs of men trained to timber in certain ways. One firm, for example, has a method of sinking pits down to sub-grade and placing supporting posts therein before excavating the main body of the material. This method might not prove satisfactory to another contracting firm, on account of not having a trained organization used to handling it.

Generally speaking, the contractor who takes the financial responsibility of the work is entitled to do it in any way he chooses, provided he does it in a way that the supervising engineer can accept as safe, from the standpoint of the public. Contractors often develop, under the spur of financial considerations, very interesting, important, and valuable new methods of shoring. For the supervising engineer, or representative of the public, to enter into the field of temporary shoring design would be on his part a trespass on the economic functions of the contractor. The proper function of the supervising engineer ends with designing the permanent work, seeing that it is placed in accordance with specifications, and that while it is being placed the work is done in such a manner that the public safety is properly served.

There are a few points in this paper which are worthy of further mention. The excellent treatment of the I-beams should be noted.

Mr. Moulton.

It will be observed that the beams are in pairs bolted together against a timber filler block, and also that the adjacent pairs are stayed with diagonal braces and steel rods with turnbuckles to resist tension. The steel members, in this manner, should gain stability and resistance against overturning.

In regard to blasting, it seems strange to a mining engineer that this should be considered as so much of a problem in connection with its effects on the shoring structure. In removing ore underground by the square set timber system, where the lower part of the face is advanced ahead of the upper part, it is not uncommon to fire from 40 to 50 shots at a round in a face 50 ft. wide. The holes may be from 2 to 10 ft. from the nearest supporting timbers, and the blast may be heavy enough to break the rock into pieces sufficiently small to be sent down the chutes and trammed out in small cars. The nearest timbers are usually protected by spiking on old planks temporarily. Instead of attempting to keep the timbers sufficiently far from the face to prevent them from being struck by rock from blasts, it is the usual mining practice to design the timbering system in such a way that posts and caps have end bearings to enable them to resist the blows of flying rock; and the connections are also designed in such a manner that the timber system as a whole is not affected if one or two posts or caps should be broken by flying rocks. This point is raised only as an interesting example of the differences in methods developed by subway contractors and mining engineers, which differences, on analysis, would probably prove to be due to material variation in the conditions under which the work is to be carried on, or to the labor and organization available.

Mr. Fiesel.

RICHARD A. FIESEL,* ESQ.—The speaker desires to express his appreciation of this excellent paper. The complexity of the subway structure, the condition of the rock, and the magnitude of the work on Section 13 demanded the attention of intrepid men of sound engineering ability. The structure designed by, and built under the direction of, Messrs. Lueder and Wilson was put to a severe test on one occasion when a blast displaced a tower. The roadway dropped less than 13 in. An examination showed that the superstructure was intact. The adaptability of this structure to the conditions encountered on that part of Section 13 where it was used proved good judgment in its selection.

The speaker is not of the opinion that a comprehensive design of street supports, made by competent engineers, will better guarantee the safety of the city streets, if bid on as a part of each contract, because the shifting of the responsibility for the safety of these structures from the contractor to the City is likely to have an unde-

* New York City.

Mr. Fiesel. sirable effect, and may lead to conditions fraught with danger. The authors' type of structure is a striking example of what the contractors and their engineers are doing to guarantee the safety of the city streets. Knowing that efforts such as these are being made, it would not be wise for the City to launch out on the proposed venture.

There are other considerations: the type of temporary structures used by the contractor for shoring the sides of an excavation, or supporting the roadway, or both, is sometimes an eleventh hour tentative selection. Any type selected must be adapted to the contractor's equipment and method of operation. Sometimes the tentative design is materially modified as a result of studies made as excavation progresses. The contractor may enlarge his equipment or may entirely change his method of operation. The inclusion in the contract of anything affecting this freedom of selection, modification, or operation, would place an unjust burden on the contractor.

A comprehensive design of roadway supports, included in the contract as a separate item, would render the City liable for the safety of the structure, wherefore, in order to protect itself against liabilities arising from possible failures, the City would be compelled to submit designs, similar, in all probability, to those in use to-day, but embodying a factor of safety unwarranted by the conditions encountered, thus adding enormously to the cost of the permanent structure.

Mr. O'Brien. J. H. O'BRIEN,* M. AM. SOC. C. E.—This is a most valuable and timely paper. The speaker, however, is inclined to doubt the practicability and wisdom of the main conclusion if he is correct in understanding that the authors mean to put the design of a scheme of temporary supports wholly on the shoulders of the Public Service Commission.

Under such an arrangement, it is not improbable that one more obstacle would be added for the ingenuity of the expert contractor to overcome in planning the execution of his work.

The scheme would undoubtedly aid an inexperienced contractor, but inexperienced contractors should not be permitted to undertake such work, irrespective of their financial ability.

An engineer charged with the responsibility of awarding contracts in private work would not risk an award to a company, the officers or members of which had had no prior experience in the work.

The speaker recalls an instance where a public commission undertook to design a shut-off dam which formed the basis of part of the contractor's tender.

When the time came to build it, the contractors refused to take the responsibility of carrying out the commission design, and submitted an alternate which was not acceptable to the commission. At the

* New York City.

suggestion of the contractors, an eminent consulting engineer was called in. He prepared a design acceptable to both parties, and it was carried out without difficulty. Mr. O'Brien.

The speaker is fully mindful of the responsibility of the public body in the case of temporary street supports, but is inclined to recommend that it be a mandatory stipulation of contracts for subway work that the contractor shall obtain the formal approval of the commission for a system of supports designed by him, based on specifications defining loading, permissible unit stresses, and other essential restrictions of good design which should be embodied in the specifications of the contract on which he tenders.

B. C. COLLIER,* M. AM. SOC. C. E.—This paper is very pertinent at this time, when the subject of underground railways is being considered in so many sections of this and other cities; and, especially under such conditions, the conclusion reached by the authors warrants the fullest consideration. Mr. Collier.

They have shown that their design of street supports, in rock cuts similar to those found in New York City, is most excellent and one that warrants the fullest consideration for any locality; but it should be borne in mind that a design applicable in one locality may not be utilizable in another, due to sub-surface structures, or for other reasons.

The authors, however, in their main conclusion, call attention to a point which, in the speaker's opinion, is almost axiomatic, in their suggestion that the engineers of the commissions (or other supervising bodies) call for bids based on previously prepared plans for shoring the cuts and supporting the surface. This suggestion will probably be objected to by numerous engineers who will cite that:

First.—It will not allow the contractor the full scope of his ingenuity or the benefit of his experience.

Second.—It will relieve the contractor from his responsibility, and throw the burden on the engineer.

The speaker does not believe that either of these objections (if raised) can stand the "spotlight" of careful analysis. Certainly, no one would do anything to prescribe the ingenuity of the contractor or his engineers, and the speaker cannot see in what way he would be prescribed by having such suggestions made.

The speaker does not go as far as the authors and agree that the engineers should submit a definite plan of shoring, for experience has shown that in most subway cuts each portion of the work is a question unto itself, and one that demands more or less individual treatment; but he believes that suggestions based on careful study of

* New York City.

Mr. Collier.

the situation could be made, which, in themselves, would call for a unit of measurement and a basis for payment. By submitting such proposed plans, and specifying that the contractor might make any modifications or substitutions which would pass the scrutiny and meet the approval of the engineers, the latter would not deny the contractor the benefit of his ingenuity or experience, because he could then compare the proposed scheme with any other he might have in view and govern his bid accordingly, thereby giving him a still further advantage over his less experienced competitor. It must be borne in mind that the preparation of plans, for a work of such large proportions as the usual subway contract, involves the study and thought of numerous men for months, and that the contractor has only a few days in which to analyze the conditions and prepare a bid under which a contract is signed, which contract demands a rigid fulfillment on the contractor's part of all the obligations he has to assume in so doing. The speaker holds, therefore, that anything that can be done by the engineers to prevent the contractor from having to take unusual risks is to the advantage of all parties, and that the establishment of a unit price for such work as "timbering" would be a "long step" toward that end. The engineers would also be directly benefited in that they would have more freedom of action to make such changes or increases as they might feel that the situation demanded, by being able to offer direct compensation to the contractor for doing this work.

It must be borne in mind, also, that all information furnished to the contractor relative to sub-surface conditions is very indefinite; that a plan which might be supposed to govern all conditions would be totally inadequate for the developed conditions; and that any system which demands that the contractor assume all responsibility and financial obligation for this is neither fair nor equitable, and should be changed. The speaker believes that the more closely a specification approximates to a condition under which the risk that a contractor takes is reduced to a minimum, the more satisfactory will be the relationship between the parties working under such specification.

In answer to the second objection, the speaker is of the opinion that, by suggesting or approving a plan for timbering, the engineer does not assume the responsibility. Certainly, no one can infer that the engineer, by withholding such written approval, but still giving his tacit acquiescence, has relieved himself from the obligation that should fall on him, of seeing that the work is carried on in a safe and workmanlike manner. How, then, when a plan which is thought out by both parties, and approved by them, is carried out, will the contractor be relieved from his obligations; for it must be remembered that it is his money which is at stake, and that it is he who must "pay the piper" in case of an accident? Why should it be the policy of an engineer to "hide behind the contractor's skirts", and run to

cover in case of trouble? Accidents will occur, under the most careful supervision, and the more direct the responsibility an engineer is made to feel, the more precaution will be taken; and as the contractor is not made to feel that he has to bear the financial burden of this precaution, he can offer no objection. This brings into consideration another feature suggested by the authors, namely, that of payment for work done outside of neat lines. One of the greatest sources of contention between engineers and contractors on the New York Subway has been the matter of payment for rock excavation beyond the neat lines, and for the concrete which the contractor is required to place to fill such spaces. Of course, it is true that this same subject has been a "bone of contention" on work from time immemorial, and has demanded numerous solutions in various sections, but this in no wise modifies the conditions that exist in New York City, and should be called to the attention of engineers who will perhaps have similar conditions elsewhere. As pointed out by the authors, probably no city in the country is founded on rock of so treacherous a character as New York, and the speaker believes it is inadvisable to provide for payment, other than the usual overbreakage allowances, unless it is thought that there will be slides of a nature similar to those occurring in New York. In such cases, however, he thinks that the specifications should be drawn so as to provide compensation to the contractor for the expense involved under such circumstances by fixing a unit price for payment, which would be large enough to afford a degree of financial relief, but not sufficiently large to create an incentive on the part of the contractor to cause such a condition deliberately. This would be a further advance toward the end desired by the speaker to see specifications drawn so as to reduce the gambling feature of the contract to a minimum. Mr. Collier.

Lazarus White,* Assoc. M. Am. Soc. C. E.—This paper opens up several topics for discussion. Of course, the important thing is always safety—"safety first". The fundamental feature of this paper is that it advocates the design, by the contractor, of a system which will guarantee absolute safety to the public; and, at the same time, other conditions are given. The system, it is stated, must also be economical as regards cost of erection. Is it possible in a system of temporary supports to have absolute safety and economy at the same time? Mr. White.

The usual subway contractor cannot allow much more than $1 per yd. for supports. If he did he would never get a contract, and if he tried to obtain absolute safety, the cost would be prohibitive, as that is something which cannot be realized.

In the speaker's opinion, all that can be done is to adopt a design which has a reasonably safe margin, and then trust to rigid supervision on the part of the engineers and contractors to keep it safe.

* New York City.

Mr. White.

No system of support, however designed, can be safe in itself. It is placed, day and night, by various gangs of timbermen, who have to be trusted to a great extent. They have to work under a series of difficult conditions, at various heights, and on underground structures, all of which require modifications in the supports. No predetermined design could be made to fit these conditions, and, if the contractors tried to make a rigid design, they would get into trouble.

The next point is the supposition that, if the timbering were designed in advance by the engineers who prepare the contract, that is, the engineers of the public service commission, greater safety—the speaker will not say "absolute safety"—could be secured, and the contractors would be bidding on the same basis; but, the speaker agrees with most of those who have discussed this matter, that this would not be desirable.

These contracts are prepared in great haste, and the commission engineers have not the time to design these timbering systems to meet the varying conditions of the different contracts; and if they were designed, the speaker does not believe that the contractor could follow them absolutely, at least, he would ask for changes, and then he would have the responsibility.

As a matter of fact—the speaker does not know whether it is intended—it seems to be an inference that the timber systems used in other structures have been erected by carpenters and various foremen. The speaker knows that such is not the case. For the last 10 or 15 years most of the contractors have had engineers, and very competent ones—members of this Society—who have designed their timber systems and looked after them. Thus, most of the timber systems were designed by competent engineers; and it makes very little difference whether they were or were not in the employ of the State.

The paper lays great emphasis on the benefit of getting out rock in large sizes, as this is considered to be safer and more economical. On that point there is a fair difference of opinion. The firm with which the speaker is connected put in bids on most of the main subway contracts, and examined the various methods of rock excavation, visiting most of the places where work was under way. The impression gained was that in those cases where it had been planned to break up the rock into small pieces (about two-man size) the material was being handled more expeditiously, and more economically than those which handled the rock in large sizes. Although the speaker's firm is at work on only one subway contract which has no rock excavation, its members have been in charge of rock excavation and open-cut aqueduct and tunnel excavation, and therefore may fairly claim to be familiar with rock excavation methods.

To handle large rocks requires very large appliances, and there is always a risk of break-downs. There is no difficulty in breaking

up large rocks in the cut. With rotating drills they are very easily drilled, and then can be shot, either with the main shots or at other times. Mr. White.

The paper states that the rock shown on a truck, in Fig. 15, weighs 25 tons, but a calculation seems to indicate that it could not have weighed more than 10 tons.

The speaker believes that the most valuable part of the paper is that which deals with longitudinal supports. In that he agrees with the authors. Such supports are of enormous value in subway construction, in safeguarding it to a very large extent, and they introduce a great element of flexibility; but they are not by any means new, or peculiar to this one job, for they have been used for the last 10 years or longer, and, in fact, patents have been granted for supports of this type.

In the speaker's work he has also used longitudinal supports in the timber system, but they are quite different from those described in the paper. Longitudinal supports are all right, but it cannot be said that entire reliance can be placed on them, because the intersection of the cut is left unprotected. There is a street crossing about every 200 ft., and this requires as much support as the main street, because there are found pipes, conduits, etc.; so that, for these places at least, transverse supports are necessary.

Rock slides are a great element of danger, but there is something peculiar about a rock slide, it sometimes requires very little support to prevent it. A slight transverse support might prevent it from sliding. There is an enormous difference between nothing and some support.

A point is made of the fact of the permanency of steel compared with timber. It is hard to conceive of a timber system that would not last the length of a subway job without deteriorating; otherwise it would be unfit for use in the first instance.

The authors express the hope that blasting may be eliminated, as being a source of danger. It is a source of danger, of course, but there is about as much chance of eliminating blasting from subway rock work as of eliminating powder from war.

A. B. Lueder,* M. Am. Soc. C. E., and W. J. R. Wilson,† Esq. (by letter).—The writers are much gratified by the interest and appreciation with which this paper has been received and by the discussions. They do not concur with some ideas brought out by the discussion which do not concern street supports directly. Messrs. Lueder and Wilson.

Referring to Mr. Moulton's remarks, the writers are pleased with his general approval of the paper and interested in the point of view of a mining man, although they believe that the general practice of excavating rock in a mine is not applicable to subway excavation.

* Morristown, N. J.
† Brooklyn, N. Y.

Messrs. Lueder and Wilson.

The paper was presented in order to illustrate a method of street support which had been used successfully and had fulfilled the conditions which it was designed to meet. The writers stated expressly that this system was not designed to take care of possible slides in the side, but so that the street supports would not be endangered by a side slide or by blasting. On Section 13 side slides were not always taken care of as it is believed they should have been, but every effort was made to eliminate them by careful supervision of the timbering and blasting. Their danger was minimized by carrying the underpinning to solid rock which had no tendency to slide, or by cantilevering the buildings off rock which was likely to slide, while depending on the tower supports in the center of the cut and the continuous beams to support the street.

The writers think that the question of extensive side slides should be considered in the design of the permanent work. They believe that, as a prime consideration of design, the street supports should be made safe. Any system of street support which depends on the stability of rock in the sides presents a great element of danger. Transverse supports cannot be put in until excavation is made. If such supports were used, the same street loads (in such a case as on Section 13 with a transverse width of 60 ft.) would have to be carried on longer spans than if the spans were longitudinal. The supports on the unstable sides of the cut would be unsafe, but those actually used rested on the bottom in the center of the cut, and were obviously on safe rock. Where the cut was narrow, transverse supports were used, but the street was still supported on continuous longitudinal beams.

The question of side slides and the danger therefrom will always be pertinent to the subject of subway excavation in New York City, but it would seem to be good practice to take the street support design out of the realm of this danger, and the writers treated the subject in this way. The extensive side slide which would bring danger to the system, as used on Section 13, would carry away with it buildings and any transverse system possible on that section. A fairly large side slide that did occur left the street supporting system unaffected. In a wide cut, such as on Section 13, the houses are practically on the edge of the cut; they will be in danger before any system of timbering can be put in, and necessarily must be taken care of first. This was done by underpinning carried to rock which investigation showed to be sound and without seams. This method of underpinning is expensive to the city, and any method of permanent construction which would guarantee safety of all side-walls would eliminate much of the underpinning cost.

There seems to be no pertinency in Mr. Moulton's suggestions as to the use of a mining system in which the lower part of the face is

Messrs. Lueder and Wilson.

kept in advance of the upper part. This would do in a tunnel, but not when there is a street overhead. It is generally agreed by experienced engineers that the open quarry method is, on the whole, the cheapest yet devised for rock cuts, and ideal subway excavation approaches that method. The writers believe that in the present system of subway building, the design of the permanent work and the shoring methods used by the contractors do not provide thoroughly against the wholesale side-slide danger referred to in the discussions. They think, however, that under the present method of bidding, the contractor is not justified in providing for such a contingency. Luckily, there has not yet been such a slide, but any engineer who has actually supervised subway construction in deep rock cuts in New York City realizes that there is a possibility that it will occur. If it does, it will certainly mean the destruction of buildings and probably great loss of life. There would follow investigations and undoubtedly mandatory legislation compelling the supervising authorities to adopt measures for absolute safety. If such measures were framed by lawyers and politicians, it might greatly increase the cost of subway building. The contractor cannot be expected, as before stated, to adopt a fixed system of supporting the side rock, whether or not it appears to be dangerous, and no amount of supervision will stop a slide, or always foresee it; but the writers believe that the design of permanent work can be modified so that it can be made to take care of any possibility of an extensive slide, and at the same time give the contractor ample opportunity to use up-to-date methods for fast excavation and construction.

The City, of course, pays for the design and the erection of the contractor's temporary supports, and also for the final permanent construction. There seems to be no reason why a permanent design should not provide more flexibility and, to a large extent, be made to take the place of a contractor's temporary system. By being designed beforehand and bid on as part of the contract, it would provide safety to the public.

The writers admit that the contractor's loads are heavy, but they are under his control, and they certainly do not overrun red flags. Mr. Moulton's reference to excavation in pits—which is a soft ground proposition—would not apply of course to a rock section. In subway work workmen are trained in their especial trades as blasters, carpenters, etc., and it is the man who uses them who needs to be experienced. As has been stated, the writers do not consider it a safe system to allow workmen to build up a patch-work design; it should not be changeable at their whims.

Referring to Mr. Fiesel's remarks, that a comprehensive design of temporary roadway supports would add enormously to the cost of the permanent structure, does not the present form of contract pay the

Messrs. Lueder and Wilson.

contractor for just such a design, and is it not the office of the Public Service Commission to see that he uses a safe design? In their experience with the Public Service Commission the writers certainly found no desire to allow a contractor to make money at the expense of public safety. The point is, though, and to that extent the writers thoroughly agree with Mr. Fiesel, that the City will have to pay a great deal more for its subway construction, if it has to pay for doing the same thing twice. That is one reason they advocate the inclusion, in the design of the permanent work, of features enabling it to fulfill for the contractor the purpose of a thoroughly safe side-wall and street-supporting system on which to bid. The writers do not intend to say that the City should design an elaborate temporary structure for the contractor, apart and separate from the design of the permanent work. They had in mind a method of construction which might have been used on Section 13, which would have provided against the danger of side slides, and have also largely answered the purpose of the street-supporting system used. This scheme, however, would have had to be part of the permanent work; the contractor could not use it separately. If, though, it had been part of the contract, it would have eliminated completely any chance of large scale disaster due to the erection of an imperfect temporary system by a contractor. The paper did not bring out very clearly the exact idea of the writers in advocating pre-designed street supports. They agree, to a certain extent, with those who discussed it, in believing that it would not be advisable to have a rigid system of temporary shoring designed by the Public Service Commission and incorporated in the contract. The public, however, should be secured by the design on which the contractor bids, as brought out by Mr. O'Brien. There is no question in the mind of any engineer experienced in subway rock work that there is a possible danger from an extensive rock slide, as referred to by Messrs. Moulton and Collier.

It is believed that the present permanent design of the subway can be modified so that it will provide against any large side slide, take the place of much of the contractor's present temporary system, and yet not add to the cost of the work to the City. With such a design, the permanent work will cost probably more for the materials used, but it will insure safety, and will make excavation and underpinning costs much less. Every contractor then would be able to figure more closely, and would have to figure on doing the work safely, because the design would provide for his doing so, and it would also provide ample leeway for ingenuity in the methods used and leave him free in his own sphere.

Mr. Collier's reference to payment for excavation beyond the neat lines for rock and concrete will interest all contractors.

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Paper No. 1366

CHEMI-HYDROMETRY AND ITS APPLICATION TO THE PRECISE TESTING OF HYDRO-ELECTRIC GENERATORS*

By Benjamin F. Groat, M. Am. Soc. C. E.

With Discussion by Messrs. G. Bertram Kershaw, George W. Fuller, W. D. Peaslee, W. L. Waters, Robert E. Horton, Thomas H. Wiggin, W. S. Richmond, and Benjamin F. Groat.

Synopsis.

This paper is divided into five parts. A summary of each part (except Part IV) will be found on pages 955, 1006, 1136, and 1231.

Chemi-hydrometry, more explicitly defined in Section 6, relates to the measurement of quantities of fluids by chemical methods.

Introduction.

Owing to the large bonus and liquidated damages clauses of the contract considered in this paper, it was apparent that unusual precision was necessary in executing the tests to determine the performance and efficiency of the new hydro-electric equipment. Besides this, there were other equally cogent reasons for making the tests as precise as possible with such methods as sound engineering practice and field methods could furnish. Among these may be mentioned the strong probability of large future purchases of hydro-electric equipment, and the concomitant necessity for accurate methods of testing, coupled with the fact that there had not yet been developed

* Presented at the meeting of December 15th, 1915.

any method for measuring the volume of large quantities of turbulently flowing water, which would furnish convincing results reliable to within 1% of the true quantity.

The uncertainty attending the measurement of large volumes of flowing water by methods heretofore used, under the most favorable circumstances and according to the best information at hand, is about as follows:

Moving screen.....	½ to 1%
Weirs	1%
Current meters.....	2 to 8%, and even higher.

The last case supposes that the meters have been rated in the usual manner in still water, and that this rating is presumed to remain unchanged during the measurements. Current meters under these circumstances have been known by the writer to be in error by more than 100% of the true velocity.

So far as the writer is aware, his tests of turbines in 1911* were the first in which any attempt was ever made to correct the still-water ratings of current meters. During these tests it was shown that screw meters under-register and cup meters over-register, and the discrepancy thus arising was found to be 8% as an average, the particular types of meter involved being the larger sizes of the Haskell and Price patterns. At certain points in the races corrections exceeding 15% were found to be necessary.

In view of these facts, and the fact that the contract under consideration specified the Haskell meter as one of the instruments for testing, it was decided to use the chemical method in parallel with three types of meter: the Haskell, Ott, and small Price. The Ott meter is of the screw type, and the small Price is a more accurate instrument than the larger type.

Ratings of all the meters were made at the Naval Tank at the University of Michigan, and curves were constructed showing the relative divergences of the ratings for given amounts of oscillation which were supposed to represent disturbance or "turbulence" of flow. Both transverse and longitudinal oscillations, with reference to the direction of motion of the meters, or water, were studied.

The results of the subsequent tests on the Allis-Chalmers equipment in the summer of 1914 confirm the conclusions drawn from the

* *Transactions*, Am. Soc. C. E., Vol. LXXVI, p. 819.

tests of 1911 concerning current meters, and the chemical tests, which are hereafter shown to be accurate to a fraction of 1%, prove conclusively that current-meter work in perturbed water must include corrections to be applied to the still-water rating, if accurate results are to be exacted. They also show that the methods of correcting the meter records used by the writer in 1914 are sufficiently accurate to secure results within 1 or 2%, even where corrections of 15%, or more, must be applied to records based on still-water ratings.

The chemical method had never been used in measuring large quantities of water with precision, the quantities dealt with usually being limited to a few cubic feet per second in the case of high-head plants where the turbines were fed by pipe lines. Somewhat larger volumes had been measured by Dr. Leon W. Collet, in the case of Swiss mountain torrents, and Stromeyer, in England, had measured boiler feed and some small streams at even earlier dates.

After reviewing these experiments, a plan was devised for testing large-capacity turbine units where discharges of several thousand cubic feet per second could be measured with precision. In devising this plan, the very accurate method of Dr. R. Mellet, of the University of Lausanne, Switzerland, for performing high-precision titrations was adopted, but it was found on trial that the method of unbalanced evaporations for conducting the remainder of the tests introduced a systematic error of greater or less degree, the magnitude of which in the present series of tests was about $\frac{1}{4}$ of 1 per cent.

The writer was not enthusiastic over the chemical method until he read Dr. Mellet's extremely valuable paper, presenting the basis for these exceedingly accurate titrations, and demonstrating clearly what degree of accuracy can be secured by such a procedure. Even then it was questionable as to how applicable the method would be to turbines of large capacity operating on low heads with correspondingly large discharges. The very satisfactory tests cited by Dr. Mellet, where the discharge of the Day power-plant at Vallorbe was accurately determined, would scarcely give any idea of the possible precision attainable in the proposed tests, because the discharge measured at Vallorbe was only 262 liters per sec., whereas it would now be necessary to measure 50 000 liters per sec. with all possible precision.

Especially was there much doubt as to the uniformity of mixture which could be secured, and as to the method of introducing the

chemical into the head-race. The methods of sampling were also to be determined, as well as the size of the solution tank and the proportions of the other parts of the testing equipment.

After some further study of actual tests, it was found that the method suggested by the writer in a paper which he presented before the Engineers' Society of Western Pennsylvania in December, 1913, eliminated the error above mentioned, along with a number of other errors, and that, in fact, it supplies all the elements of precision necessary for the most refined work. The formula which indicates the methods first appears in a somewhat more complicated form than necessary.* These methods have since been applied successfully by the writer in several tests in different parts of the country.

In order to show that turbine testing has been anything but an exact science in the past, it may be stated that there have been several extensive series of tests which can be regarded in no other light than as failures, in so far as decisively definite results are concerned, and that in at least one case, after an expenditure of many thousands of dollars, there was an uncertainty as to the resulting efficiency amounting to 3 per cent. Current meters were used in that test.

Turbine testing and discharge gauging have been more of an art than a science. Results have depended more for accuracy on the particular person who directed the work than on any method heretofore used. It has been a common occurrence that there has been a complete failure to interpret the data correctly, with the result that much of the work in discharge measurements has been unsatisfactory. When results have been shown to be accurate, we may rest assured that the person conducting the tests has arrived at a conclusion only by the most painstaking efforts and perseverance.

The results of the studies mentioned will be treated in five parts:

I.—History and Theory;
II.—Errors;
III.—Description of Testing Plant;
IV.—The Tests;
V.—Current Meters.

A brief summary is given at the beginning of each part.

* *Proceedings*, Engineers' Society of Western Pennsylvania, May, 1914, Vol. 30, p. 374.

PART I.

HISTORY AND THEORY.

SUMMARY.

1.—In Part I a complete theory of mixtures of salt solutions of various densities has been evolved and elaborated into a comprehensive treatise on the subject. This includes tables and diagrams of properties of the salt solution. This theory is necessary for high-precision work on account of the shrinkage in volume which takes place when two or more different salt solutions are mixed.

The term "Chemi-hydrometry" has been proposed as the name for this new branch of engineering science.

The reader's attention is called to the important fact that there is great advantage to be gained by measuring quantities of the solutions by weight rather than by volume. This eliminates in large measure the necessity for observing the temperatures of the solutions and making the corrections therefor. There is further gain in accuracy due to the fact that there is less likelihood of contaminating the samples by reason of neglect to clean the thermometer each time before it is dipped into a sample or other solution. Moreover, this method gives the real quantity which is desired in turbine testing, namely, the weight of the water passing through the water-wheels as against the volume.

A complete method has been devised for the precise testing of hydro-electric units of large capacity with a chemical injected into and mixed with the feed-water. This method is based on the original suggestion by Schloesing in France, in 1863, the method of precise titrations of weak salt solutions devised by Dr. R. Mellet, and the design and operation of chemical testing plants, as suggested in a paper by the writer before the Engineers' Society of Western Pennsylvania in December, 1913.* This paper also contains the formula which leads to the method of balanced evaporations, accomplished by special dilutions in the laboratory.

The formula for balanced evaporations is treated in full, as it applies to tests under various conditions. It leads to the elimination of all uncertainty as to the properties of the normal head-water. It indi-

* *Proceedings*, Engineers' Society of Western Pennsylvania, May, 1914.

cates and represents a definite laboratory procedure which has been outlined in accordance therewith.

The coefficient of shrinkage of volume, when two salt solutions are mixed, has been fully treated and tabulated.

Theoretically correct equations, involving concentrations only, are established.

2.—Theory and Properties of the Salt Solution.—We are indebted to Landolt and Börnstein* for the most complete tables of the densities of salt solutions of different percentages up to saturated solutions at temperatures varying from 0° to 80° cent. These tables, therefore, will be taken as the basis for the development of the theory of the salt solution as applied in the chemical method for measuring quantities of water.

As the theory which is to be presented concerns largely the concentrations of various solutions, it will be convenient to elaborate on the tables to some extent by appending a column of concentrations. It will frequently be convenient to know also the quantity, either by volume or by weight, of distilled water or of salt, contained in a solution of given concentration. Therefore, additional columns, for these and similar purposes, are appended, so as to furnish the data in Table 1, which data are limited to temperatures between 0° and 40° cent.

3.—Approximate Formulas for Weak Solutions.—As Table 1 does not extend to salt solutions of less than ½ of 1%, several equations have been devised for computing densities in terms of concentrations, or percentages, and *vice versa*. These equations are merely approximate, but cannot be far from correct, especially as regards relative errors in computing densities. The general equation for density, d_x, at temperature, x, in terms of the density, d, of distilled water, at temperature, x, and one parameter, n, depending on the temperature, is,

$$d_x = d + n_x p_x \left\{ \begin{array}{c} \text{Limited to solutions of less than } \tfrac{1}{2}\% \\ \text{in salt} \end{array} \right\} \quad \ldots\ldots\ldots\ldots (1)$$

* "Physikalisch-Chemische Tabellen." Landolt-Börnstein. Published by Julius Springer, Berlin, 1912, p. 260. See also a paper by W. J. Walker "On the Relationship between the Viscosity, Density, and Temperature of Salt Solutions." *Phil. Mag.*, February, 1914, Series 6, Vol. 27, No. 158.

TABLE 1.—ONE LITER OF SALT SOLUTION, OF VARYING DENSITY, AT DIFFERENT TEMPERATURES.

TEMPERATURE = 0° CENT.

Volume of 1 Gramme of Distilled Water = 1.000132 c.c.

Percentage of salt solution.	Density, in grammes per cubic centimeter.	Concentration, in grammes per liter.	Weight of distilled water per liter of solution, in grammes.	Volume of distilled water per liter of solution, in cubic centimeters.	Apparent volume of salt per liter of solution, in cubic centimeters.	Salt per cubic centimeter of apparent volume, in grammes.	Apparent volume occupied per gramme of salt, in cubic centimeters.
0.5	1.0037	5.0185	998.68	998.81	1.1867	4.2289	0.23646
1.0	1.0076	10.076	997.52	997.66	2.3443	4.2981	0.23266
1.5	1.0114	15.171	996.23	996.36	3.6895	4.1684	0.23990
2.0	1.0153	20.306	994.99	995.13	4.8747	4.1656	0.24006
2.5	1.0191	25.478	993.62	993.75	6.2468	4.0786	0.24519
3.0	1.0230	30.690	992.31	992.44	7.5590	4.0601	0.24630
3.5	1.0268	35.938	990.86	990.99	9.0072	3.9899	0.25063
4.0	1.0307	41.228	989.47	989.60	10.397	3.9654	0.25218
4.5	1.0345	46.552	987.95	988.08	11.922	3.9047	0.25610
5.0	1.0384	51.920	986.48	986.61	13.390	3.8775	0.25789
5.5	1.0422	57.321	984.88	985.01	14.991	3.8237	0.26153
6.0	1.0461	62.766	983.33	983.46	16.536	3.7957	0.26345
6.5	1.0500	68.250	981.75	981.88	18.120	3.7666	0.26549
7.0	1.0538	73.766	980.03	980.16	19.837	3.7186	0.26892
7.5	1.0577	79.328	978.37	978.50	21.499	3.6898	0.27101
8.0	1.0616	84.928	976.67	976.80	23.199	3.6608	0.27316
8.5	1.0654	90.559	974.84	974.97	25.030	3.6180	0.27639
9.0	1.0693	96.237	973.06	973.19	26.809	3.5897	0.27857
9.5	1.0732	101.95	971.25	971.38	28.621	3.5621	0.28073
10.0	1.0771	107.71	969.39	969.52	30.482	3.5336	0.28300
10.5	1.0810	113.50	967.50	967.63	32.372	3.5061	0.28521
11.0	1.0849	119.34	965.56	965.69	34.313	3.4780	0.28752
11.5	1.0888	125.21	963.59	963.72	36.283	3.4509	0.2977
12.0	1.0927	131.12	961.58	961.71	38.293	3.4241	0.29204
12.5	1.0966	137.07	959.53	959.66	40.343	3.3976	0.29432
13.0	1.1005	143.06	957.44	957.57	42.434	3.3714	0.29661
13.5	1.1044	149.09	955.31	955.44	44.564	3.3455	0.29890
14.0	1.1083	155.16	953.14	953.27	46.734	3.3201	0.30120
14.5	1.1123	161.28	951.02	951.15	48.854	3.3013	0.30291
15.0	1.1162	167.43	948.77	948.90	51.105	3.2762	0.30523
15.5	1.1202	173.63	946.57	946.69	53.305	3.2573	0.30700
16.0	1.1241	179.86	944.24	944.36	55.635	3.2329	0.30932
16.5	1.1281	186.14	941.96	942.08	57.916	3.2140	0.31114
17.0	1.1321	192.46	939.64	939.76	60.236	3.1951	0.31298
17.5	1.1361	198.82	937.28	937.40	62.596	3.1762	0.31483
18.0	1.1401	205.22	934.88	935.00	64.997	3.1574	0.31672
18.5	1.1441	211.66	932.44	932.56	67.437	3.1386	0.31861
19.0	1.1481	218.14	929.96	930.08	69.917	3.1200	0.32051
19.5	1.1521	224.66	927.44	927.56	72.438	3.1014	0.32243
20.0	1.1562	231.24	924.96	925.08	74.918	3.0866	0.32398
20.5	1.1602	237.84	922.36	922.48	77.518	3.0682	0.32592
21.0	1.1643	244.50	919.80	919.92	80.079	3.0532	0.32752
21.5	1.1684	251.21	917.19	917.31	82.689	3.0380	0.32916
22.0	1.1724	257.93	914.47	914.59	85.409	3.0199	0.33113
22.5	1.1765	264.71	911.79	911.91	88.090	3.0050	0.33278
23.0	1.1806	271.54	909.06	909.18	90.820	2.9899	0.33446
23.5	1.1848	278.43	906.37	906.49	93.510	2.9775	0.33585
24.0	1.1889	285.34	903.56	903.68	96.321	2.9624	0.33756
25.0	1.1972	299.30	897.90	898.02	101.98	2.9348	0.34073
26.0	1.2056	313.46	892.14	892.26	107.74	2.9093	0.34371
26.4	1.2089	319.15	889.75	889.87	110.13	2.8979	0.34507

TABLE 1.—*(Continued.)*

TEMPERATURE = 10° CENT.

Volume of 1 Gramme of Distilled Water = 1.000273 c.c.

Percentage of salt solution.	Density, in grammes per cubic centimeter.	Concentration, in grammes per liter.	Weight of distilled water per liter of solution, in grammes.	Volume of distilled water per liter of solution, in cubic centimeters.	Apparent volume of salt per liter of solution, in cubic centimeters.	Salt per cubic centimeter of apparent volume, in grammes.	Apparent volume occupied per gramme of salt, in cubic centimeters.
0.5	1.0034	5.0170	998.38	998.66	1.3444	3.7317	0.26797
1.0	1.0071	10.071	997.03	997.30	2.6988	3.7316	0.26798
1.5	1.0108	15.162	995.64	995.91	4.0903	3.7069	0.26977
2.0	1.0145	20.290	994.21	994.48	5.5186	3.6766	0.27199
2.5	1.0182	25.455	992.74	993.01	6.9840	3.6447	0.27437
3.0	1.0219	30.657	991.24	991.51	8.4864	3.6125	0.27683
3.5	1.0256	35.896	989.70	989.97	10.026	3.5802	0.27931
4.0	1.0293	41.172	988.13	988.40	11.602	3.5486	0.28179
4.5	1.0330	46.485	986.52	986.78	13.216	3.5173	0.28431
5.0	1.0367	51.835	984.86	985.13	14.866	3.4868	0.28679
5.5	1.0405	57.228	983.27	983.54	16.460	3.4767	0.28762
6.0	1.0442	62.652	981.55	981.82	18.184	3.4454	0.29024
6.5	1.0479	68.114	979.79	980.05	19.947	3.4147	0.29285
7.0	1.0517	73.619	978.08	978.35	21.652	3.4001	0.29411
7.5	1.0554	79.155	976.24	976.51	23.488	3.3703	0.29675
8.0	1.0592	84.736	974.46	974.73	25.270	3.3539	0.29822
8.5	1.0629	90.346	972.55	972.82	27.180	3.3240	0.30084
9.0	1.0667	96.003	970.70	970.96	29.038	3.3061	0.30247
9.5	1.0705	101.70	968.80	969.06	30.986	3.2874	0.30419
10.0	1.0742	107.42	966.78	967.04	32.956	3.2595	0.30680
10.5	1.0780	113.19	964.81	965.07	34.927	3.2408	0.30857
11.0	1.0818	119.00	962.80	963.06	36.937	3.2217	0.31039
11.5	1.0856	124.84	960.76	961.02	38.978	3.2028	0.31222
12.0	1.0894	130.73	958.67	958.93	41.068	3.1838	0.31416
12.5	1.0933	136.66	956.64	956.90	43.099	3.1708	0.31537
13.0	1.0971	142.62	954.48	954.74	45.259	3.1512	0.31734
13.5	1.1009	148.62	952.28	952.54	47.460	3.1315	0.31934
14.0	1.1048	154.67	950.13	950.39	49.611	3.1177	0.32075
14.5	1.1087	160.70	948.00	948.26	51.741	3.1058	0.32197
15.0	1.1125	166.88	945.62	945.88	54.129	3.0834	0.32433
15.5	1.1164	173.04	943.36	943.62	56.382	3.0691	0.32583
16.0	1.1203	179.25	941.05	941.31	58.693	3.0540	0.32744
16.5	1.1242	185.49	938.71	938.97	61.034	3.0391	0.32904
17.0	1.1281	191.78	936.32	936.58	63.424	3.0238	0.33071
17.5	1.1321	198.12	933.98	934.23	65.765	3.0125	0.33195
18.0	1.1360	204.48	931.52	931.77	68.226	2.9971	0.33366
18.5	1.1400	210.90	929.10	929.35	70.646	2.9853	0.33497
19.0	1.1439	217.34	926.56	926.81	73.187	2.9697	0.33674
19.5	1.1479	223.84	924.06	924.31	75.688	2.9574	0.33813
20.0	1.1519	230.38	921.52	921.77	78.228	2.9450	0.33956
20.5	1.1559	236.96	918.94	919.19	80.809	2.9323	0.34102
21.0	1.1600	243.60	916.40	916.65	83.350	2.9226	0.34216
21.5	1.1640	250.26	913.74	913.99	86.011	2.9096	0.34369
22.0	1.1681	256.98	911.12	911.37	88.631	2.8994	0.34489
22.5	1.1721	263.72	908.38	908.63	91.372	2.8862	0.34647
23.0	1.1762	270.53	905.67	905.92	94.083	2.8754	0.34777
23.5	1.1803	277.37	902.93	903.18	96.824	2.8647	0.34908
24.0	1.1845	284.28	900.22	900.47	99.534	2.8561	0.35013
25.0	1.1927	298.18	894.52	894.76	105.24	2.8331	0.35294
26.0	1.2011	312.29	888.81	889.05	110.95	2.8147	0.35523
26.4	1.2045	317.99	886.51	886.75	113.25	2.8079	0.35614

TABLE 1.—(*Continued.*)

TEMPERATURE = 15° CENT.

Volume of 1 Gramme of Distilled Water = 1.000874 c.c

Percentage of salt solution.	Density, in grammes per cubic centimeter.	Concentration, in grammes per liter.	Weight of distilled water per liter of solution, in grammes.	Volume of distilled water per liter of solution, in cubic centimeters.	Apparent volume of salt per liter of solution, in cubic centimeters.	Salt per cubic centimeter of apparent volume, in grammes.	Apparent volume occupied per gramme of salt, in cubic centimeters.
0.5	1.0028	5.0140	997.79	998.66	1.3419	3.7365	0.26763
1.0	1.0064	10.064	996.84	997.21	2.7932	3.6030	0.27754
1.5	1.0100	15.150	994.85	995.72	4.2805	3.5398	0.28254
2.0	1.0137	20.274	993.43	994.29	5.7058	3.5532	0.28143
2.5	1.0173	25.432	991.87	992.78	7.2651	3.5006	0.28567
3.0	1.0209	30.627	990.27	991.12	8.8625	3.4558	0.28937
3.5	1.0246	35.861	988.74	989.60	10.397	3.4492	0.28992
4.0	1.0282	41.128	987.07	987.93	12.065	3.4089	0.29335
4.5	1.0319	46.436	985.46	986.33	13.675	3.3956	0.29449
5.0	1.0355	51.775	983.72	984.58	15.415	3.3587	0.29773
5.5	1.0392	57.156	982.04	982.90	17.098	3.3428	0.29915
6.0	1.0429	62.574	980.33	981.18	18.817	3.3254	0.30072
6.5	1.0466	68.029	978.57	979.43	20.574	3.3066	0.30243
7.0	1.0503	73.521	976.78	977.63	22.367	3.2870	0.30423
7.5	1.0540	79.050	974.95	975.80	24.198	3.2668	0.30611
8.0	1.0577	84.616	973.08	973.93	26.066	3.2462	0.30805
8.5	1.0614	90.219	971.18	972.03	27.970	3.2256	0.31002
9.0	1.0651	95.859	969.24	970.09	29.912	3.2047	0.31204
9.5	1.0688	101.54	967.26	968.11	31.895	3.1836	0.31411
10.0	1.0726	107.26	965.34	966.18	33.816	3.1719	0.31527
10.5	1.0763	113.01	963.29	964.13	35.868	3.1507	0.31739
11.0	1.0801	118.81	961.29	962.13	37.870	3.1373	0.31874
11.5	1.0838	124.64	959.16	960.00	40.002	3.1158	0.32094
12.0	1.0876	130.51	957.09	957.93	42.074	3.1019	0.32238
12.5	1.0914	136.42	954.98	955.81	44.185	3.0875	0.32389
13.0	1.0952	142.38	952.82	953.65	46.347	3.0720	0.32552
13.5	1.0990	148.36	950.64	951.47	48.529	3.0571	0.32710
14.0	1.1028	154.39	948.41	949.24	50.761	3.0415	0.32878
14.5	1.1066	160.46	946.14	946.97	53.033	3.0256	0.33051
15.0	1.1105	166.58	943.92	944.74	55.255	3.0147	0.33170
15.5	1.1143	172.73	941.58	942.40	57.597	2.9968	0.33347
16.0	1.1182	178.91	939.29	940.11	59.889	2.9874	0.33474
16.5	1.1221	185.15	936.95	937.77	62.231	2.9752	0.33611
17.0	1.1260	191.42	934.58	935.40	64.603	2.9630	0.33749
17.5	1.1299	197.73	932.12	932.98	67.015	2.9505	0.33892
18.0	1.1338	204.08	929.72	930.53	69.467	2.9378	0.34039
18.5	1.1378	210.49	927.31	928.12	71.880	2.9284	0.34149
19.0	1.1417	216.92	924.78	925.59	74.412	2.9150	0.34304
19.5	1.1457	223.41	922.29	923.10	76.904	2.9050	0.34423
20.0	1.1497	229.94	919.76	920.56	79.436	2.8947	0.34546
20.5	1.1537	236.51	917.19	917.99	82.008	2.8840	0.34674
21.0	1.1577	243.12	914.58	915.38	84.621	2.8730	0.34806
21.5	1.1617	249.77	911.93	912.73	87.273	2.8619	0.34941
22.0	1.1657	256.45	909.25	910.04	89.955	2.8509	0.35077
22.5	1.1698	263.20	906.60	907.39	92.608	2.8421	0.35185
23.0	1.1739	270.00	903.90	904.69	95.310	2.8329	0.35300
23.5	1.1780	276.83	901.17	901.96	98.042	2.8236	0.35416
24.0	1.1821	283.70	898.40	899.19	100.81	2.8142	0.35534
25.0	1.1904	297.60	892.80	893.58	106.42	2.7965	0.35759
26.0	1.1987	311.66	887.04	887.82	112.18	2.7782	0.35994
26.4	1.2021	317.35	884.75	885.52	114.48	2.7721	0.36074
26.8	1.2055	323.07	882.43	883.20	116.80	2.7660	0.36153

TABLE 1.—(*Continued.*)

TEMPERATURE = 20° CENT.

Volume of 1 Gramme of Distilled Water = 1.001773 c.c.

Percentage of salt solution.	Density, in grammes per cubic centimeter.	Concentration, in grammes per liter.	Weight of distilled water per liter of solution, in grammes.	Volume of distilled water per liter of solution, in cubic centimeters.	Apparent volume of salt per liter of solution, in cubic centimeters.	Salt per cubic centimeter of apparent volume, in grammes.	Apparent volume occupied per gramme of salt, in cubic centimeters.
0.5	1.0019	5.0095	996.89	998.66	1.3420	3.7329	0.26789
1.0	1.0054	10.054	995.35	997.11	2.8893	3.4797	0.28738
1.5	1.0090	15.135	993.86	995.62	4.3729	3.4611	0.28893
2.0	1.0126	20.252	992.35	994.11	5.8926	3.4369	0.29096
2.5	1.0161	25.402	990.70	992.45	7.5455	3.3665	0.29704
3.0	1.0197	30.591	989.11	990.86	9.1373	3.3479	0.29869
3.5	1.0233	35.816	987.48	989.23	10.765	3.3271	0.30056
4.0	1.0269	41.076	985.82	987.57	12.428	3.3051	0.30256
4.5	1.0305	46.372	984.13	985.87	14.127	3.2825	0.30465
5.0	1.0341	51.705	982.40	984.14	15.863	3.2596	0.30680
5.5	1.0378	57.079	980.72	982.46	17.540	3.2542	0.30729
6.0	1.0414	62.484	978.92	980.65	19.348	3.2295	0.30965
6.5	1.0450	67.925	977.06	978.81	21.193	3.2051	0.31201
7.0	1.0487	73.409	975.29	977.02	22.980	3.1945	0.31304
7.5	1.0523	78.922	973.38	975.10	24.896	3.1701	0.31545
8.0	1.0560	84.480	971.52	973.24	26.756	3.1574	0.31671
8.5	1.0597	90.074	969.63	971.35	28.655	3.1434	0.31813
9.0	1.0633	95.697	967.69	969.39	30.681	3.1191	0.32061
9.5	1.0670	101.36	965.64	967.35	32.648	3.1046	0.32210
10.0	1.0707	107.07	963.63	965.34	34.661	3.0891	0.32378
10.5	1.0744	112.81	961.59	963.29	36.705	3.0734	0.32537
11.0	1.0781	118.59	959.51	961.21	38.789	3.0573	0.32708
11.5	1.0819	124.42	957.48	959.18	40.822	3.0479	0.32810
12.0	1.0856	130.27	955.33	957.02	42.976	3.0312	0.32990
12.5	1.0894	136.18	953.23	954.91	45.090	3.0202	0.33111
13.0	1.0931	142.10	951.00	952.69	47.314	3.0033	0.33296
13.5	1.0969	148.08	948.82	950.50	49.498	2.9916	0.33427
14.0	1.1007	154.10	946.60	948.28	51.722	2.9794	0.33564
14.5	1.1045	160.15	944.35	946.03	53.976	2.9671	0.33708
15.0	1.1083	166.24	942.06	943.73	56.270	2.9543	0.33849
15.5	1.1122	172.39	939.81	941.48	58.524	2.9456	0.33949
16.0	1.1160	178.56	937.44	939.10	60.898	2.9321	0.34105
16.5	1.1199	184.78	935.12	936.78	63.222	2.9227	0.34215
17.0	1.1237	191.03	932.67	934.32	65.676	2.9087	0.34380
17.5	1.1276	197.33	930.27	931.93	68.081	2.8985	0.34501
18.0	1.1315	203.67	927.83	929.48	70.525	2.8879	0.34627
18.5	1.1355	210.07	925.43	927.07	72.929	2.8805	0.34717
19.0	1.1394	216.49	922.91	924.55	75.454	2.8692	0.34853
19.5	1.1433	222.94	920.36	921.99	78.008	2.8579	0.34991
20.0	1.1473	229.46	917.84	919.47	80.533	2.8493	0.35097
20.5	1.1513	236.02	915.28	916.90	83.097	2.8403	0.35208
21.0	1.1553	242.61	912.69	914.31	85.692	2.8312	0.35321
21.5	1.1593	249.25	910.05	911.66	88.336	2.8216	0.35441
22.0	1.1633	255.93	907.37	908.98	91.021	2.8118	0.35565
22.5	1.1674	262.66	904.74	906.34	93.656	2.8045	0.35657
23.0	1.1714	269.42	901.98	903.58	96.421	2.7942	0.35788
23.5	1.1755	276.24	899.26	900.85	99.146	2.7863	0.35891
24.0	1.1796	283.10	896.50	898.09	101.91	2.7779	0.35998
25.0	1.1879	296.98	890.92	892.50	107.50	2.7626	0.36198
26.0	1.1963	311.04	885.26	886.83	113.17	2.7484	0.36384
26.4	1.1996	316.69	882.91	884.48	115.52	2.7414	0.36477
26.8	1.2030	322.40	880.60	882.16	117.84	2.7359	0.36551

TABLE 1.—(*Continued.*)

Temperature = 40° Cent.

Volume of 1 Gramme of Distilled Water = 1.00782 c.c.

Percentage of salt solution.	Density, in grammes per cubic centimeter.	Concentration, in grammes per liter.	Weight of distilled water per liter of solution, in grammes.	Volume of distilled water per liter of solution, in cubic centimeters.	Apparent volume of salt per liter of solution, in cubic centimeters.	Salt per cubic centimeter of apparent volume, in grammes.	Apparent volume occupied per gramme of salt, in cubic centimeters.
0.5	0.9957	4.9785	990.72	998.47	1.5311	3.2516	0.30754
1.0	0.9991	9.991	989.11	996.84	3.1562	3.1655	0.31590
1.5	1.0025	15.038	987.46	995.18	4.8161	3.1224	0.32026
2.0	1.0060	20.120	985.88	993.59	6.4104	3.1386	0.31861
2.5	1.0094	25.235	984.16	991.86	8.1388	3.1006	0.32252
3.0	1.0129	30.387	982.51	990.20	9.8038	3.0995	0.32263
3.5	1.0163	35.570	980.78	988.40	11.601	3.0661	0.32615
4.0	1.0198	40.792	979.01	986.66	13.336	3.0588	0.32693
4.5	1.0233	46.048	977.25	984.89	15.106	3.0483	0.32805
5.0	1.0267	51.335	975.36	982.99	17.008	3.0183	0.33131
5.5	1.0302	56.661	973.54	981.15	18.848	3.0062	0.33265
6.0	1.0337	62.022	971.68	979.28	20.723	2.9929	0.33412
6.5	1.0373	67.424	969.88	977.46	22.540	2.9913	0.33430
7.0	1.0408	72.856	967.94	975.51	24.487	2.9753	0.33610
7.5	1.0443	78.322	965.98	973.53	26.468	2.9591	0.33794
8.0	1.0478	83.824	963.98	971.51	28.486	2.9426	0.33983
8.5	1.0514	89.369	962.03	969.55	30.446	2.9353	0.34068
9.0	1.0550	94.950	960.05	967.56	32.442	2.9268	0.34167
9.5	1.0586	100.57	958.09	965.52	34.478	2.9169	0.34288
10.0	1.0621	106.21	956.89	963.37	36.635	2.8991	0.34493
10.5	1.0658	111.91	958.89	961.35	38.651	2.8954	0.34538
11.0	1.0694	117.63	951.77	959.21	40.787	2.8840	0.34674
11.5	1.0730	123.40	949.60	957.03	42.974	2.8715	0.34825
12.0	1.0766	129.19	947.41	954.82	45.181	2.8594	0.34973
12.5	1.0803	135.04	945.26	952.65	47.348	2.8521	0.35062
13.0	1.0840	140.92	943.08	950.45	49.545	2.8443	0.35158
13.5	1.0877	146.84	940.86	948.22	51.782	2.8357	0.35264
14.0	1.0914	152.80	938.60	945.94	54.060	2.8265	0.35380
14.5	1.0951	158.79	936.31	943.63	56.368	2.8170	0.35498
15.0	1.0988	164.82	933.98	941.28	58.716	2.8071	0.35624
15.5	1.1026	170.90	931.70	938.99	61.014	2.8010	0.35702
16.0	1.1063	177.01	929.29	936.56	63.443	2.7901	0.35841
16.5	1.1101	183.17	926.93	934.18	65.821	2.7829	0.35934
17.0	1.1139	189.36	924.54	931.77	68.230	2.7753	0.36032
17.5	1.1177	195.60	922.10	929.31	70.689	2.7671	0.36140
18.0	1.1216	201.89	919.71	926.90	73.098	2.7619	0.36207
18.5	1.1254	208.20	917.20	924.37	75.628	2.7529	0.36325
19.0	1.1293	214.57	914.73	921.88	78.117	2.7468	0.36406
19.5	1.1332	220.97	912.23	919.36	80.636	2.7403	0.36492
20.0	1.1371	227.42	909.68	916.79	83.206	2.7332	0.36587
20.5	1.1410	233.90	907.10	914.19	85.806	2.7259	0.36685
21.0	1.1449	240.43	904.47	911.54	88.457	2.7180	0.36791
21.5	1.1489	247.01	901.89	908.94	91.057	2.7127	0.36864
22.0	1.1529	253.64	899.26	906.29	93.708	2.7067	0.36945
22.5	1.1569	260.30	896.60	903.61	96.389	2.7005	0.37030
23.0	1.1609	267.01	893.89	900.88	99.120	2.6938	0.37122
23.5	1.1649	273.75	891.15	898.12	101.88	2.6870	0.37216
24.0	1.1690	280.56	888.44	895.39	104.61	2.6820	0.37286
25.0	1.1772	294.30	882.90	889.80	110.20	2.6706	0.37445
26.0	1.1855	308.23	877.27	884.13	115.87	2.6601	0.37592
26.4	1.1888	313.84	874.96	881.80	118.20	2.6552	0.37663
26.8	1.1922	319.51	872.09	879.51	120.49	2.6518	0.37711

TABLE 1.—(*Continued.*)

Temperature = 60° Cent.

Volume of 1 Gramme of Distilled Water = 1.01705 c.c.

Percentage of salt solution.	Density, in grammes per cubic centimeter.	Concentration, in grammes per liter.	Weight of distilled water per liter of solution, in grammes.	Volume of distilled water per liter of solution, in cubic centimeters.	Apparent volume of salt per liter of solution, in cubic centimeters.
0.5	0.9869	4.9345	981.97	998.71	1.2920
1.0	0.9903	9.903	980.40	997.11	2.8872
1.5	0.9937	14.906	978.79	995.48	4.5176
2.0	0.9971	19.942	977.16	993.82	6.1815
2.5	1.0005	25.012	975.49	992.12	7.8799
3.0	1.0039	30.117	973.78	990.39	9.6140
3.5	1.0073	35.256	972.04	988.62	11.383
4.0	1.0107	40.428	970.27	986.82	13.185
4.5	1.0141	45.634	968.47	984.98	15.022
5.0	1.0175	50.875	966.62	983.11	16.894
5.5	1.0210	56.155	964.84	981.30	18.704
6.0	1.0244	61.464	962.94	979.35	20.646
6.5	1.0279	66.814	961.09	977.47	22.527
7.0	1.0314	72.198	959.20	975.56	24.444
7.5	1.0348	77.610	957.19	973.51	26.490
8.0	1.0383	83.064	955.24	971.52	28.477
8.5	1.0418	88.553	953.25	969.50	30.500
9.0	1.0453	94.077	951.22	967.44	32.559
9.5	1.0489	99.646	949.25	965.44	34.561
10.0	1.0524	105.24	947.16	963.31	36.691
10.5	1.0559	110.87	945.08	961.14	38.857
11.0	1.0595	116.54	942.96	959.04	40.962
11.5	1.0631	122.26	940.84	956.88	43.119
12.0	1.0666	127.99	938.61	954.61	45.387
12.5	1.0702	133.78	936.42	952.39	47.614
13.0	1.0738	139.59	934.21	950.14	49.862
13.5	1.0775	145.46	932.04	947.93	52.069
14.0	1.0811	151.35	929.75	945.60	54.398
14.5	1.0848	157.30	927.50	943.31	56.686
15.0	1.0884	163.26	925.14	940.91	59.086
15.5	1.0921	169.28	922.82	938.55	61.446
16.0	1.0958	175.33	920.47	936.16	63.836
16.5	1.0995	181.42	918.08	933.73	66.267
17.0	1.1032	187.54	915.66	931.27	68.728
17.5	1.1070	193.72	913.28	928.85	71.149
18.0	1.1107	199.93	910.77	926.30	73.701
18.5	1.1145	206.18	908.32	923.81	76.193
19.0	1.1183	212.48	905.82	921.26	78.736
19.5	1.1221	218.81	903.29	918.69	81.309
20.0	1.1259	225.18	900.72	916.08	83.923
20.5	1.1298	231.61	898.19	913.50	86.496
21.0	1.1336	238.06	895.54	910.81	89.191
21.5	1.1375	244.56	892.94	908.16	91.835
22.0	1.1414	251.11	890.29	905.47	94.531
22.5	1.1453	257.69	887.61	902.74	97.256
23.0	1.1492	264.32	884.88	899.97	100.03
23.5	1.1532	271.00	882.20	897.24	102.76
24.0	1.1572	277.73	879.47	894.46	105.54
25.0	1.1652	291.30	873.90	888.80	111.20
26.0	1.1733	305.06	868.24	883.04	116.96
26.4	1.1765	310.60	865.90	880.66	119.34
26.8	1.1798	316.19	863.61	878.33	121.67

TABLE 1.—(*Continued.*)

TEMPERATURE = 80° CENT.

Volume of 1 Gramme of Distilled Water = 1.02899 c.c.

Percentage of salt solution.	Density, in grammes per cubic centimeter.	Concentration, in grammes per liter.	Weight of distilled water per liter of solution, in grammes.	Volume of distilled water per liter of solution, in cubic centimeters.	Apparent volume of salt per liter of solution, in cubic centimeters.
0.5	0.9771	4.8855	972.21		
1.0	0.9805	9.805	970.70	998.84	1.1646
1.5	0.9839	14.758	969.14	997.24	2.7626
2.0	0.9873	19.746	967.55	995.60	4.3966
2.5	0.9908	24.770	966.03	994.04	5.9648
3.0	0.9942	29.825	964.38	992.33	7.6678
3.5	0.9976	34.916	962.68	990.59	9.4078
4.0	1.0010	40.040	960.96	988.82	11.182
4.5	1.0044	45.198	959.20	987.01	12.991
5.0	1.0079	50.395	957.50	985.26	14.737
5.5	1.0113	55.622	955.68	983.38	16.617
6.0	1.0148	60.888	953.91	981.57	18.434
6.5	1.0182	66.183	952.02	979.62	20.384
7.0	1.0217	71.519	950.18	977.73	22.273
7.5	1.0251	76.882	948.22	975.71	24.293
8.0	1.0286	82.288	946.31	973.75	26.254
8.5	1.0320	87.720	944.28	971.65	28.345
9.0	1.0355	93.195	942.30	969.62	30.378
9.5	1.0390	98.705	940.30	967.55	32.446
10.0	1.0425	104.25	938.25	965.45	34.550
10.5	1.0460	109.83	936.17	963.31	36.690
11.0	1.0495	115.44	934.06	961.14	38.862
11.5	1.0530	121.10	931.90	958.92	41.084
12.0	1.0565	126.78	929.73	956.67	43.327
12.5	1.0601	132.51	927.59	954.48	45.519
13.0	1.0636	138.27	925.33	952.16	47.845
13.5	1.0671	144.06	923.04	949.80	50.201
14.0	1.0707	149.90	920.80	947.49	52.506
14.5	1.0743	155.77	918.53	945.16	54.842
15.0	1.0778	161.67	916.13	942.69	57.311
15.5	1.0814	167.62	913.78	940.27	59.730
16.0	1.0850	173.60	911.40	937.82	62.179
16.5	1.0886	179.62	908.98	935.33	64.669
17.0	1.0923	185.69	906.61	932.89	67.107
17.5	1.0959	191.78	904.12	930.33	69.670
18.0	1.0995	197.91	901.59	927.73	72.273
18.5	1.1032	204.09	899.11	925.18	74.825
19.0	1.1069	210.31	896.59	922.58	77.418
19.5	1.1105	216.55	893.95	919.87	80.134
20.0	1.1142	222.84	891.36	917.20	82.799
20.5	1.1179	229.17	888.73	914.49	85.506
21.0	1.1217	235.56	886.14	911.83	88.171
21.5	1.1254	241.96	883.44	909.05	90.949
22.0	1.1292	248.43	880.78	906.31	93.686
22.5	1.1329	254.90	878.00	903.45	96.547
23.0	1.1367	261.44	875.26	900.63	99.366
23.5	1.1405	268.02	872.48	897.77	102.23
24.0	1.1443	274.63	869.67	894.88	105.12
25.0	1.1520	288.00	864.00	889.05	110.95
26.0	1.1597	301.52	858.18	883.06	116.94
26.4	1.1628	306.98	855.82	880.63	119.37
26.8	1.1660	312.49	853.51	878.25	121.75

By determining the values of n for the various temperatures and then inserting these values in Equation (1) along with those of d for distilled water, there may be derived,

$$\left.\begin{aligned} d_0 &= 0.999868 + 0.772\,p_0 \\ d_{10} &= 0.999727 + 0.740\,p_{10} \\ d_{15} &= 0.999126 + 0.730\,p_{15} \\ d_{20} &= 0.998230 + 0.720\,p_{20} \\ d_{40} &= 0.992240 + 0.690\,p_{40} \end{aligned}\right\} \text{Limited to solutions of less than } \tfrac{1}{2}\% \text{ in salt} \quad \ldots\ldots(2)$$

which, for percentages, may be rewritten as follows:

$$\left.\begin{aligned} 100\,p_0 &= 129.53d - 129.52 \\ 100\,p_{10} &= 135.14d - 135.10 \\ 100\,p_{15} &= 136.99d - 136.87 \\ 100\,p_{20} &= 138.89d - 138.64 \\ 100\,p_{40} &= 144.93d - 143.80 \end{aligned}\right\} \text{Limited to solutions of less than } \tfrac{1}{2}\% \text{ in salt} \quad \ldots\ldots(3)$$

There may also be written:

$$\left.\begin{aligned} d_0 &= 0.499934 + \sqrt{0.249935 + 0.000772\,C} \\ d_{10} &= 0.499864 + \sqrt{0.249865 + 0.000740\,C} \\ d_{15} &= 0.499563 + \sqrt{0.249565 + 0.000730\,C} \\ d_{20} &= 0.499115 + \sqrt{0.249116 + 0.000720\,C} \\ d_{40} &= 0.496120 + \sqrt{0.246135 + 0.000690\,C} \end{aligned}\right\} \text{Limited to solutions of less than } \tfrac{1}{2}\% \text{ in salt.} \quad \ldots(4)$$

in which C is the concentration.

4.—For purposes of engineering and computation, a diagram plotted to a convenient scale will greatly facilitate determinations of the values of the quantities involved in Table 1 and the foregoing formulas without the necessity for calculations and interpolations. Plate XXXIII exhibits this plotting, the scale being sufficiently large to enable a reading of density to be made with great precision. This diagram, however, is not so accurate when it is required to ascertain the value of the concentrations of salt solutions in terms of the percentage. In order to ascertain concentrations with a sufficient degree of accuracy, Plates XXXIV and XXXV have been specially prepared. These diagrams give the relation between percentage and concentration with a high degree of precision when the temperature is between 10 and 30° cent., and may be used even beyond these limits. It is scarcely neces-

PLATE XXXIII.
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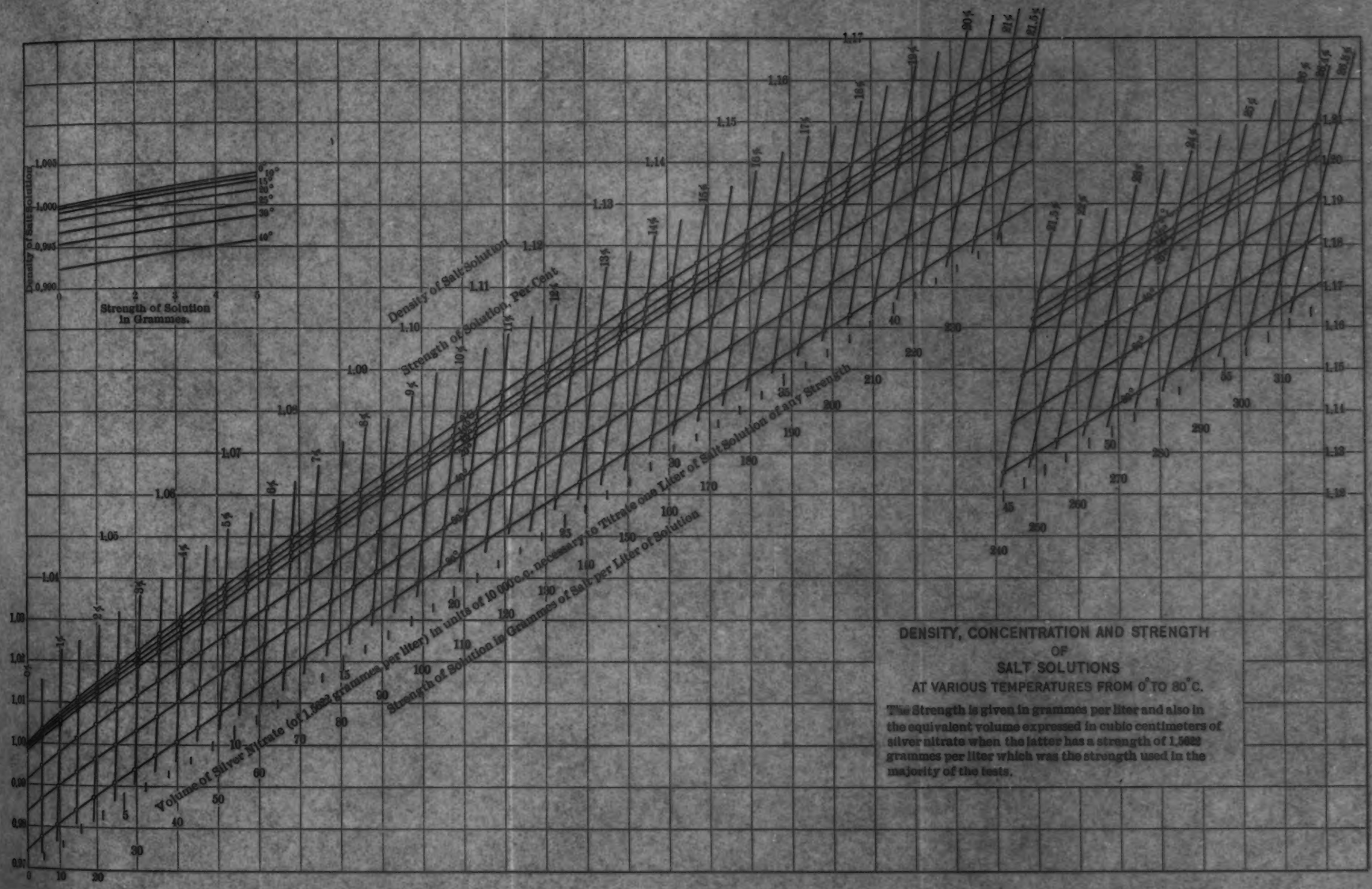

DENSITY, CONCENTRATION AND STRENGTH OF SALT SOLUTIONS AT VARIOUS TEMPERATURES FROM 0° TO 80° C.

The Strength is given in grammes per liter and also in the equivalent volume expressed in cubic centimeters of silver nitrate when the latter has a strength of 1.5822 grammes per liter which was the strength used in the majority of the tests.

sary to make any further explanation, as the plotting will be sufficiently well understood.

5.—*Volumetric Shrinkage.*—When two salt solutions of different densities are mixed, the volume of the resultant mixture is less than the sum of the volumes of the two components before mixing. Table 1 and Plates XXXIII, XXXIV, and XXXV show this, as illustrated by the following example, the figures for which have been taken from Table 1:

Of a ¼% solution, 100 liters are mixed with 1 liter of 25% solution at 20° cent. Find the percentage of the mixture, the density, and final volume, at the given temperature.

Solution:

	Kilogrammes of solution.	Kilogrammes of NaCl.
1 liter, 25% solution.....	1.1879	0.29698
100 liters, ¼% solution.....	100.19	0.50095
Totals	101.3779	0.79793

$$\text{New percentage of the mixture} = \frac{0.79793}{101.3779} = 0.78708.$$

The density for this percentage is 1.0039.

Therefore the final volume of the mixture is 101.378 ÷ 1.0039 = 100.984. Thus, the shrinkage of volume is 101 − 100.984 = 0.016 liter.

This illustration shows that there is a shrinkage of volume, and that, for a ratio of mixture of 100 to 1, the relative shrinkage is very small.

Take another case, where the ratio of mixture is, say, only 1 to 1. Let 1 liter of 25% solution be mixed with 1 liter of distilled water at 20° cent. Find the percentage of the mixture at the given temperature.

Solution:

	Kilogrammes of solution.	Kilogrammes of salt.
1 liter, 25% solution.....	1.1879	0.29698
1 liter, distilled water.....	0.9982	0.00000
Total weight	2.1861	0.29698

New percentage = 0.29698 ÷ 2.1861 = 13.585

Corresponding density = 1.0975 (Table 1).

Resulting volume = 2.1861 ÷ 1.0975 = 1.99189

Thus, the shrinkage of volume is 2 − 1.992 = 0.008 liter.

Although this result is less than the former one, the relative error is much larger, being about 0.4% of the entire volume and 0.8% of the volume of the distilled water with which the strong solution was mixed. In this case the error is also 0.8% of the volume of the strong salt solution which is diluted with the distilled water.

These illustrations point to the fact that the shrinkage is not great, but may be sufficiently large to be taken into account when errors of less than 1% are to be computed.

6.—Chemi-Hydrometry.—As the method of measuring quantities of liquids by chemical procedure has not yet received a distinctive name, it may be appropriate to suggest one. The word "Chemi-hydrometry" seems to describe very exactly the nature of the process, but has three features which seem to be more or less unfavorable to its adoption.

The word seems to be too long to recommend itself. It is a compound of words derived from different languages, which is not good form. The last component has been used almost exclusively in reference to determinations of density and specific gravity, and is not so well known as relating to measurements of quantities of liquids.

However, until a better name is suggested, Chemi-hydrometry will be used by the writer as referring to the body of rules, formulas, and processes connected with the measurement of quantities of liquids by chemical processes.

7.—Mixtures of Solutions.—When two salt solutions of different percentages are mixed at a given temperature, the mixture, when at the same temperature, will not have a volume equal to the sum of the two volumes mixed, but will be more or less deficient. In consequence of this shrinkage of volume, it is necessary to develop a theory of mixtures before volumes can be determined by chemi-hydrometry with certainty.

By "ratio of mixture" is meant the ratio of the quantity of the weaker of two solutions to the quantity of the other with which it is mixed. These quantities may be measured in units of volume or units of weight, according to circumstances or convenience. The symbol for this ratio will be f for units of weight, and r for units of volume.

By "ratio of dilution" is meant the ratio of the quantity resulting from the mixture of two salt solutions of different percentages to the quantity having the higher percentage which has been mixed with

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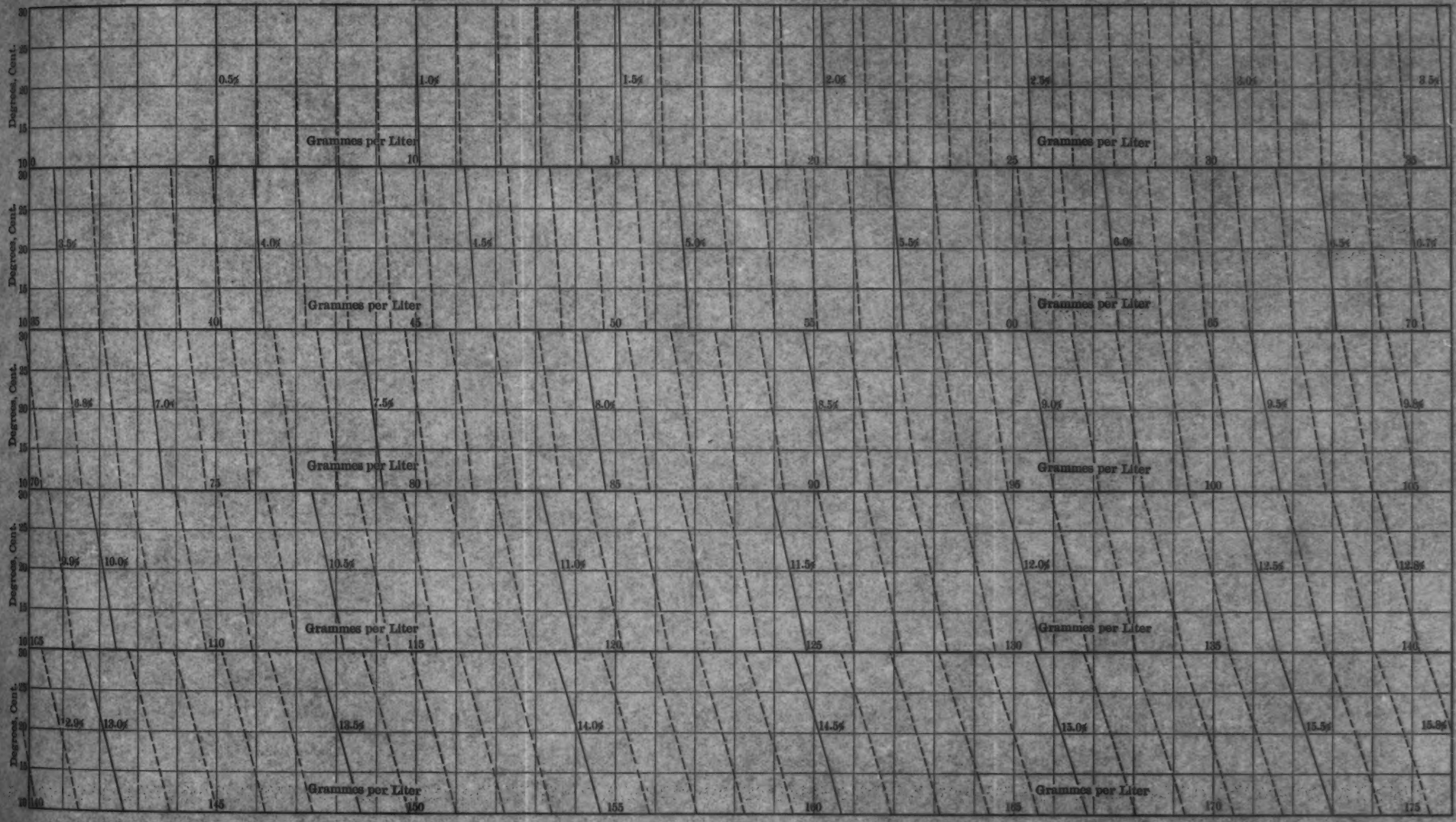
CONCENTRATION AND STRENGTH OF SALT SOLUTIONS BETWEEN 10° AND 30° CENTIGRADE.



the other. The symbol for this ratio will be F for units of weight, and R for units of volume.

When a solution is referred to as "being of a certain percentage" it is meant that this percentage of the total weight of the solution is anhydrous chemical, or, in the case of salt solutions, simply dry salt.

By "strength of solution" will be meant either the percentage of the solution, or its equivalent, the ratio of the weight of salt in solution to the total weight of the solution. This latter ratio will be indicated by p. The percentage of a solution, therefore, is equal to $100\ p$.

By "concentration of a solution" is meant the weight of the chemical in solution per unit of volume; for example, the number of grammes of salt per liter of solution. The symbol, c, represents the concentration of a solution.

When quantities of solution are measured by weight, W and w will be used to represent these weights, and Q or q will represent volumes.

When quantities of silver nitrate, or other standard solution, are measured from a volumetric burette, in executing a titration, the symbol, t, will denote the volume measured, usually in cubic centimeters.

The quantity of a sample of solution to be titrated will be indicated by V in the case of measurement by volume and by S in the case of measurement by weight. Thus, $t \div V$ and $t \div S$ are volumes of silver nitrate consumed per unit quantity of sample, based on the titration, t.

The titration per unit quantity of sample will be denoted by v, so that equations of the form $v = t \div V$ and $v = t \div S$ will occur.

3.—Volume, Strength, Concentration, and Density of a Mixture of Several Known Salt Solutions.—Let the several known solutions be indicated by consecutive subscripts, as 1, 2, 3. . . .n. Then the weight of the solution, x, is W_x, its strength is p_x, and the weight of the contained salt is $W_x\ p_x$. Therefore,

$$\text{Total weight of a mixture of } n \text{ solutions} = \Sigma\, W_x \quad \ldots\ldots (4)$$

$$\text{Total weight of salt in the } n \text{ solutions} = \Sigma\, W_x\, p_x$$

$$\text{Strength of the mixture of } n \text{ solutions} = p = \frac{\Sigma\, W_x\, p_x}{\Sigma\, W_x} \quad \ldots\ldots (5)$$

The reader will not fail to observe that this formula is the same as that for a weighted average, which, of course, is the same as that for the average arm of several forces, or the "arm of gravity."

From the resulting value of p, it will be easy, with Plate XXXIII, to determine the density, d, of the mixture of the n solutions; and with Plates XXXIII, XXXIV, or XXXV, when great accuracy is required, the concentration, c, may be found.

Or, having found the density from Plate XXXIII, the concentration may be computed, thus:

$$\text{Concentration} = c = pd \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

It will also be seen at a glance that

$$Q = \frac{\Sigma W_x}{d} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

9.—Mixture of Two Solutions.—For the ordinary purposes of chemi-hydrometry, mixtures of two solutions, one of which is strong and the other comparatively weak, are of most interest. By omitting subscripts for the strong solution, using the subscript, 1, for the weaker component solution and 2 for the resultant mixture, the following special cases of Equations (5) and (7) may be written at once:

$$\left.\begin{aligned} p_2 &= \frac{wp + W_1 p_1}{w + W_1} \\ Q_2 &= \frac{w + W_1}{d_2} \end{aligned}\right\} \left\{\begin{array}{l} \text{Simple letters refer to strong salt solution. Subscript, 1, refers to 2d component of mixture. Subscript, 2, refers to resulting mixture.} \end{array}\right\} \ldots\ldots (8)$$

Equations (8) are rigorous, and may be used to determine the volume of a mixture of two solutions, in terms of the weights and strengths of the constituents, d_2 becoming known as soon as p_2 has been determined, as has been explained.

It is sometimes required to find the strength of a resultant solution in terms of the ratio of mixture, or of the ratio of dilution of two solutions, thus:

$$p_2 = \frac{p + f p_1}{1 + f} \qquad f = \frac{W_1}{w} \quad \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

and

$$p_2 = \frac{(p - p_1) + F p_1}{F} \qquad \left\{\begin{array}{l} F = W_2 \div w \\ W_2 = w + W_1 \end{array}\right\} \quad \ldots\ldots\ldots\ldots (10)$$

PLATE XXXV.
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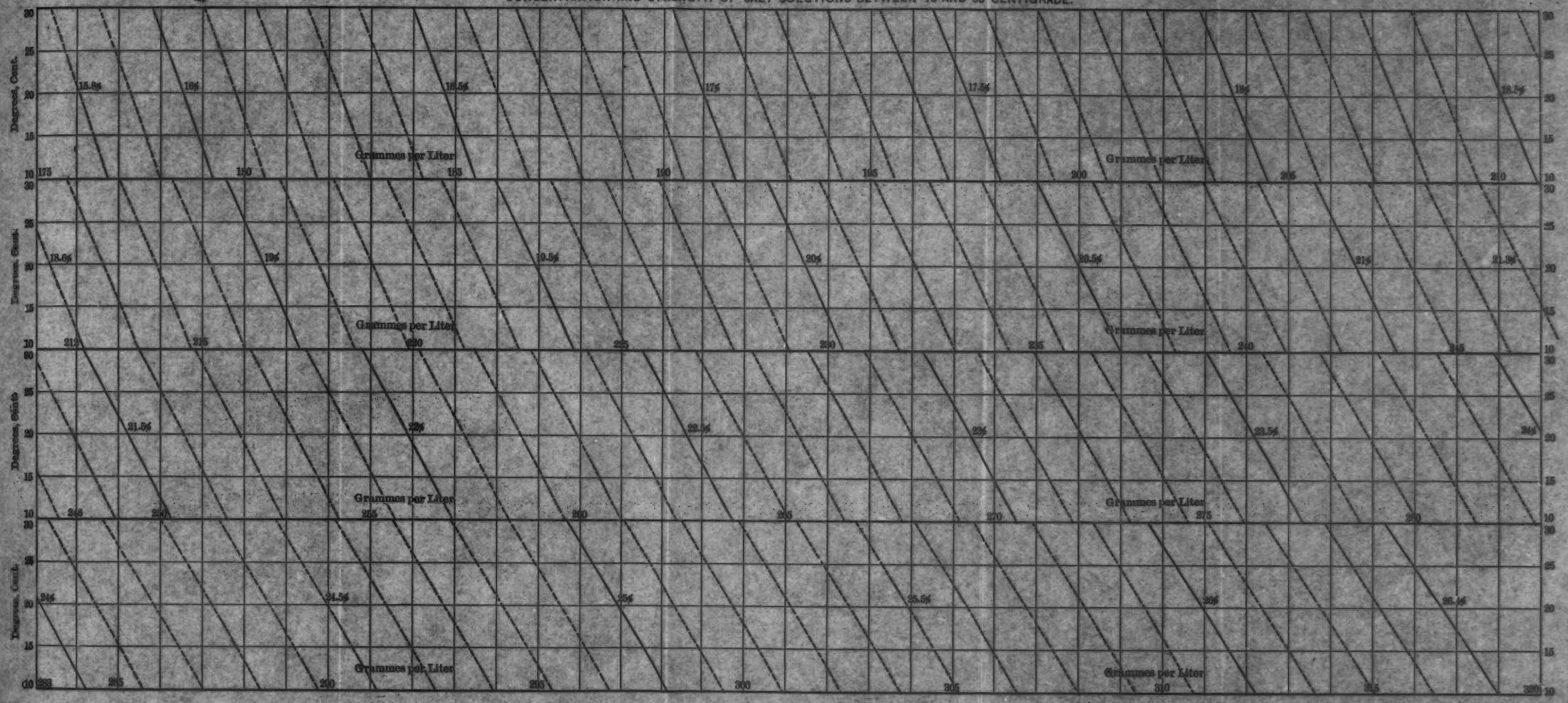
CONCENTRATION AND STRENGTH OF SALT SOLUTIONS BETWEEN 10° AND 30° CENTIGRADE.



10.—Equations Based on Weights and Strengths Only.—It does not appear that any recognition has been given to the fact that, if the weights of the dosing solution and samples for titration are observed, it will be possible to compute the discharge of a turbine in units of weight, the only units really necessary for a determination of efficiency, without any temperature corrections. The titrations of the samples, of course, are measured in units of volume of silver nitrate, the same as in all cases.

It is very clear, therefore, that much of the labor of computation may be saved by using units of weight in determining quantities of solution and sizes of samples, rather than units of volume. This, however, makes somewhat more trouble in calibrating the chemical tank, as the weight of solution consumed depends on the density, where it is the volume taken from the tank that is measured. On the other hand, this is not so serious a matter as the calculations necessary to ascertain the corrections to be applied to titrations, volumes, and densities.

It follows from this that the ideal method of measuring the dosing solution is by weight, and not by volume. Where this is possible, the method of weights should be adopted unqualifiedly. The importance of this, in the case considered herein, was not realized in time to arrange the testing equipment for the method of weights. However, as the theory for this method is simple, and promises much for the future, it will be given first.

It should be observed, first, that the chemical method determines quantities, that is, weights or volumes of water, or other liquid. Consequently, it is applicable for finding the capacities of tanks and reservoirs, as the volume of such a receiver is equal to the volume of liquid which fills it. Indeed, the capacity of a reservoir, up to any given elevation, can be found immediately by simply ascertaining the quantity of liquid it contains when filled up to that elevation.

This can be done in two ways, one or the other of which may be found the more convenient. The tank may be first filled to the given elevation with the normal water from a water supply, after which an appropriate quantity of salt solution is mixed with the normal water; or, the tank being filled nearly to the given elevation with normal water, salt solution may be added and mixed until the mixture

rises to that elevation. In the first case it is required to find the volume of the normal water before introducing salt, and in the second case it is required to find the volume of the mixture.

In both cases the necessary data are the quantity of salt solution mixed with the normal water and the respective strengths of the salt solution, the normal water, and the mixture. The strengths are determined by chemical analyses, usually by titrations of the samples.

The first of these two cases corresponds to determining the quantity of water passing through a turbine when the salt is injected into the tail-water discharged from the turbine, and samples of the mixture are taken at a point still farther down stream. The second case corresponds to a determination of the quantity of mixture discharged from a turbine when the salt is injected into the head-water, and the mixture is sampled either in the head-race, before it reaches the turbine, or in the tail-race, after passing through the turbine. The difficulties of sampling are discussed in Section 55 *et seq.* See also Sections 58, 59, and 84.

11.—It will facilitate comprehension of the equations to remember that letters without subscripts relate to salt solutions, Subscript 1 relates to normal water, and Subscript 2 relates to the resultant mixture. Thus, w is a quantity of salt solution considered either as a whole, in the case of calibrating a tank, or as a rate of quantity; that is, a quantity per unit of time, as in the case of a turbine test. It may be well, also, to remember that the normal water may contain salt or other chemical initially, thus complicating the equations to a certain extent. Therefore:

Weight of salt in normal water $= W_1 p_1$

" " " " salt solution $= w p$

" " " " resultant mixture $= W_2 p_2$

From which

$$W_1 p_1 + w p = W_2 p_2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

But

$$W_2 = W_1 + w$$

$$W_1 \div w = f$$

$$W_2 \div w = F$$

Therefore,

$$\left.\begin{aligned} f + 1 &= F, \\ f p_1 + p &= F p_2, \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

Thus,
$$\left.\begin{aligned} f &= \frac{F p_2 - p}{p_1} \\ F &= \frac{f p_1 + p}{p_2} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

Again, by eliminating first F, and then f, from Equations (12),

and
$$\left.\begin{aligned} f &= \frac{p - p_2}{p_2 - p_1} \\ F &= \frac{p_1 - p}{p_2 - p_1} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (14)$$

Equations (14) give the ratio of mixture and the ratio of dilution in terms of the strengths of the three solutions involved. With these ratios may be calculated at once the volumes of the normal head-water and resultant mixture, thus

and
$$\left.\begin{aligned} W_1 &= wf \\ W_2 &= wF \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

Now, if the end points of the titrations are in error by the same relative amount, we should have

$$\left.\begin{aligned} p &= Ev \\ p_1 &= Ev_1 \\ p_2 &= Ev_2 \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

as each unit of volume of silver nitrate would correspond to a definite quantity of salt. Consequently, we may write at once, noting that E is a common factor,

and
$$\left.\begin{aligned} W_1 &= w\frac{v - v_2}{v_2 - v_1} \\ W_2 &= w\frac{v - v_1}{v_2 - v_1} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

v being the titrations per kilogramme, or other unit of weight, reduced from actual titrations of samples in the laboratory.

It must be carefully observed that this substitution of v for p throughout an equation of chemi-hydrometry is permissible only where the error introduced is either inappreciable or entirely removed by correcting the titrations when seriously affected. The real nature of the errors committed by such substitutions will become more apparent with further study of the subject.

12.—Special Laboratory Procedure and Special Dilutions.—For the present it will suffice to state that the writer eliminates this error by resort to a special laboratory process, the principle involved being as follows:

To have two or more titrations in error by the same amount, and therefore by the same relative amount, the samples must be of exactly the same size and just alike in all respects; the standard solution and the indicator must each have the same strength in the two or more cases; and, in fact, the samples and titrations in the several cases must be just alike in all other respects as regards the conditions under which the operations take place.

The truth of this proposition, being axiomatic, needs no comment. The application of the principle is effected by making a special dilution in the laboratory which is to have the same ratios of mixture and dilution as has the mixture of salt solution and normal feed-water in the actual test. Of course, the same salt solution and normal feed-water must be used in the laboratory as on the test.

This procedure cannot be accomplished without advance knowledge of the actual ratio of dilution. In any case, it will always be possible to have a more or less approximate value for this ratio, and the approximate value may be used in making up preliminary special dilutions, from which a close approximation to the true ratio may be computed by the methods immediately to follow, thus leading to a second approximation, which, in most cases, will be sufficiently exact. In fact, it will be found in most actual tests that the true value of the ratio will be known in advance with sufficient closeness to enable the making of satisfactory special dilutions.

It will seldom chance to be the case, however, that the ratios of mixture and dilution actually used in the laboratory will prove to be equal to the true ratios corresponding. It is necessary, therefore, to distinguish carefully between the actual ratios, samples, and titrations and the special ratios, samples, and titrations of the laboratory dilutions. A prime, therefore, will be affixed to the letters and symbols pertaining to the special dilutions; thus, r', R', and w' are, respectively, the ratio of mixture, ratio of dilution, and weight of salt solution used in making up a special dilution.

13.—Assume, then, that a test has actually been made, that samples of percentages, p and p_2, have been taken, and that w has been

measured. Further, that special dilutions have been prepared, and that the quantities, f', F', w', p', and p_2', have also been observed. There may then be written, from Equations 14, Section 11, the following relations:

$$\left.\begin{aligned} \frac{F}{f} &= \frac{p - p_1}{p - p_2}, \text{ for the actual test,} \\ \text{and} \quad \frac{f'}{F'} &= \frac{p - p_2'}{p - p_1}, \text{ for the special dilutions.} \end{aligned}\right\} \quad \ldots\ldots\ldots (18)$$

It may be helpful here to remark that p and p_1 are the same in both preceding equations, as the same salt solution and the same normal feed-water have been used in the laboratory and on the actual test. Further, the unknown, p_1, may now be eliminated from Equations (18), thus:

$$\frac{Ff'}{fF'} = \frac{p - p_2'}{p - p_2} = 1 + \frac{p_2 - p_2'}{p - p_2} \quad \ldots\ldots\ldots (19)$$

Now put

$$\left.\begin{aligned} x &= \frac{p_2 - p_2'}{p - p_2} \\ \text{and} \quad y &= \frac{p_2 - p_2'}{p - p_2'} \end{aligned}\right\} \quad \ldots\ldots\ldots (20)$$

from which

$$\frac{x}{y} = \frac{1}{1 - y} = 1 + x \quad \ldots\ldots\ldots (21)$$

Therefore,

$$\frac{Ff'}{fF'} = 1 + x, \text{ or}$$

$$1 + \frac{1}{f} = \left(1 + \frac{1}{f'}\right)(1 + x) = 1 + \frac{1}{f'} + \frac{F'x}{f'} \quad \ldots\ldots\ldots (22)$$

as $F = f + 1$, and $F' = f' + 1$, by Equations (12), Section 11.

From Equation (22) may be written at once

$$f = \frac{f'}{1 + F'x}, \text{ and } \left(x = \frac{p_2 - p_2'}{p - p_2}\right) \quad \ldots\ldots\ldots (23)$$

Again,

$$\frac{fF'}{Ff'} = \frac{1}{1 + x} = 1 - y \quad \ldots\ldots\ldots (24)$$

or

$$1 - \frac{1}{F} = \left(1 - \frac{1}{F'}\right)(1 - y) \quad \ldots\ldots\ldots (25)$$

which by analogy gives at once

$$F = \frac{F'}{1 + f'y}, \text{ and } \left(y = \frac{p_2 - p_2'}{p - p_2'}\right) \quad \ldots\ldots\ldots (26)$$

If it is required to have equations in terms of either the rate of mixture or rate of dilution, they may be written out as follows, by simply putting $F - 1$ for f, or $f + 1$ for F, thus:

Method of special dilutions. Equations based on weights of samples.

$$\left.\begin{aligned} f &= \frac{f'}{1 + (f' + 1)\,x} \\ f &= \frac{F' - 1}{1 + F'\,x} \\ F &= \frac{f' + 1}{1 + f'\,y} \\ F &= \frac{F'}{1 + (F' - 1)\,y} \end{aligned}\right\} \left\{\begin{aligned} \left(x = \frac{p_2 - p_2'}{p - p_2}\right) \\ \left(y = \frac{p_2 - p_2'}{p - p_2'}\right) \end{aligned}\right\} \quad (27)$$

14.—It will not always be advisable to eliminate p_1 in this manner. Frequently, it will be advisable to determine p_1 by Equations (14), from the known ratio of mixture, or ratio of dilution, used in the preparation of the special dilutions, and then determine F, or f, by a second application of Equations (14), with the resulting value of p_1 introduced.

An advantage secured by this procedure arises when the normal feed-water for the turbine runs constant in salt content. In this case the value of p_1, reduced to standard conditions, should be the same in all the tests, if care has been taken to keep the silver nitrate at the same standard strength. An error in the laboratory process is then made apparent by a discrepancy among the corrected values of p_1.

The laboratory procedure is exactly the same as before, the only difference being in the method of calculation, which is sufficiently indicated by the following equations, derived from Equations (13) or (14).

By the method of "Special Dilutions."

$$\left.\begin{aligned} p_1 &= \frac{F'\,p_2' - p}{f'} \quad \text{(a check on uniformity of normal head-water)} \\ f &= \frac{p - p_2}{p_2 - p_1} \\ \text{or, } F &= \frac{p - p_1}{p_2 - p_1} \end{aligned}\right\} \quad \ldots(28)$$

15.—Object of the Special Dilutions.—The influence of the special dilutions toward eliminating the errors of titration may now be more fully explained.

An inspection of Equations (14), Section 11, will show at a glance that, if all the strengths of the solutions as determined by the titrations are in error by the same relative amount, the ratios of mixture and dilution, and therefore the quantities of liquid to be measured, will, nevertheless, be correctly determined by the formulas.

If the percentages, p, p_1, p_2, however, are not in error by the same relative discrepancy, then an error of greater or less moment will be introduced in the calculations, and, as a consequence, they are based on fallacious data. It should be observed here that p is relatively much larger than p_2 and p_1, and that p_2 may be any number of times larger than p_1. Thus, in the case of the tests to be discussed, the salt solution used was near the saturation point, but the normal canal water carried only about 15 mg. of salt per liter. The quantity of salt injected into the head-water was from 3 to 4 times as much as the water contained initially.

The corresponding values of v, v_2, and v_1, therefore, were about 540 000, 110, and 25 c.c. per liter of the respective solutions, when the silver nitrate was of a concentration of about 1.56 grammes per liter of distilled water. The respective values of 100 p, 100 p_2 and 100 p_1, therefore, were about 24.5%, 0.00592%, and 0.00134%, and c, c_2, and c_1, were about 290, 0.0591, and 0.0134 grammes per liter of the corresponding sample.

Table 2 facilitates a comparison of these figures.

TABLE 2.—COMPARISON OF SALT AND SILVER NITRATE EQUIVALENTS, AND STRENGTHS AND DENSITIES OF SALT SOLUTION, HEAD-WATER, AND TAIL-WATER SAMPLES INVOLVED IN THE TESTS.

$AgNO_3$ solution contains about 1.56 grammes per liter of distilled water.

Quantities compared.	Salt solution.	Tail-water.	Head-water.
Cubic centimeters of silver nitrate per liter of sample	540 000	110	25
Ditto per kilogramme of sample	456 200	110.2	25
Percentage strength of solution (100p)	24.50	0.00592	0.001342
Densities	1.1537	0.9984	0.9984
Concentration, in grammes per liter	290	0.0591	0.0134

Thus, by Equations (14), Section 11, the ratio of dilution by weight will be:

$$F = \frac{24.50 - 0.00134}{0.00592 - 0.00134} = \frac{24.50}{0.00458} = 5\,349.3 \ldots\ldots (29)$$

or, for every pound of salt solution injected into the head-race there were, as an average, 5 349 lb. of mixture flowing through the turbine.

Now the percentages, $100\ p_1 = 0.00134$, and $100\ p_2 = 0.00592$, are both in error relative to $100\ p = 24.50$, as the titrations were not of samples which were alike in all respects. For example, the titration for p is made by first diluting 10 c.c.* of salt solution to 1 liter and then placing 10 c.c. of this dilution in a casserole for the actual titration, which, therefore, would be about 54 c.c. of silver nitrate, the ten-thousandth part of what it would be for a whole liter of salt solution. On the other hand, the titrations of the normal head-water and of the tail-water are made by evaporating 500 c.c. of sample to about 10 c.c. in a casserole before making the titrations. Thus, these samples have been reduced by evaporation, whereas the salt solution sample has been increased in size by diluting with distilled water. Moreover, the concentration of the tail-water sample, and consequently its titration, are each about 4½ times what they are for the normal head-water, thus rendering p_2 and p_1 relatively inconsistent with each other as well as with p.

Under the conditions cited, it can be shown that the titration of normal head-water should be decreased about 0.14 c.c. for each ½-liter sample, a little more than 1%; and the titration of the tail-water, the ½ liter evaporation titrating so nearly equal to the titration of the salt solution, namely, about 55 c.c., needs scarcely any correction. Therefore, a more correct value for the ratio of dilution will be

$$F = \frac{24.5}{0.00592 - 0.001325} = 5\ 332 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (30)$$

which is seen to differ from the uncorrected ratio by about ⅓ of 1%, which, relatively, was too large an error to pass without notice in the turbine tests. The object of the special dilutions is to eliminate this error without the necessity for determining the corrections to be applied to the titrations.

16.—Function of the Special Dilutions.—The operation of the special dilutions to eliminate the errors of titration may be illustrated by an application of any of Equations (27) to the problem of the preceding paragraph.

* The method of weights was not used in the tests in question. Hence volumes instead of weights are given here.

First, let it be observed that the errors of titration can affect only x and y of the equations, because the results of the titrations enter only these quantities, and f' or F' are determined independently by the relative weights of the samples. Consequently, the effect of an error can be discovered by ascertaining the effect on x, or y, and then introducing the erroneous value of x, or y, in the corresponding equation.

Let S, S_2, and S'_2 be the weights of the samples at the times they are actually titrated. Let ρ_2, and ρ'_2 be the ratios of evaporation of the tail-water and special dilutions, that is, the ratio of the measured weight of a sample to its weight when ready for titration after the evaporation. In order to have the titrations as nearly alike as possible, the dilution of the salt solution and evaporations of the tail-water and special dilutions must be proportioned so that the actual samples titrated will be as nearly alike as possible. This will mean that the actual weights titrated and the strengths of these samples must be the same, and, therefore, their volumes must be the same. Thus, if ρ is the ratio of dilution of the salt solution with distilled water, c_s the concentration of the silver nitrate, and e the actual equivalent weight of salt per unit weight of silver nitrate consumed in titration, we must have,

$$\left.\begin{aligned} p = v\, c_s\, e &= \rho \frac{t}{S} c_s\, e \\ p_2 = v_2\, c_s\, e_2 &= \frac{t_2}{\rho_2 S_2} c_s\, e_2 \\ p_2' = v_2'\, c_s\, e_2' &= \frac{t_2'}{\rho_2' S_2'} c_s\, e_2' \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (31)$$

If the laboratory procedure is properly arranged, R' will be the same as R, so that the titrations for t_2 and t'_2 will be exactly alike, and, therefore, $t_2 = t'_2$, $\rho_2 = \rho'_2$, $S_2 = S'_2$, and $e_2 = e'_2$, but e will not necessarily be equal to either e_2 or e'_2, owing to the dilution of the salt solution with distilled water instead of normal head-water, and to other obvious differences among the conditions under which the operations on the salt solution and the other two kinds of samples take place.

However, we may write

$$\left.\begin{aligned} S &= S_2 = S_2' \\ e_2 &= e_2' \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (32)$$

and $\rho_2 = \rho_2'$

With Equations (31) and (32) we may substitute in the values of x and y to obtain

$$\left.\begin{aligned} x &= \frac{p_2 - p_2'}{p - p_2} = \frac{v_2 - v_2'}{v - v_2} = \frac{t_2 - t_2'}{T - t_2} \\ y &= \frac{p_2 - p_2'}{p - p_2'} = \frac{v_2 - v_2'}{v - v_2'} = \frac{t_2 - t_2'}{T - t_2'} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots (33)$$

where

$$T = \rho\, \rho_2 \frac{e}{e_2} t$$

which shows that, when $t_2 = t'_2$, the samples having been made exactly alike, $x = 0$ and $y = 0$, whether there is any error in t, that is, in T, or the contrary. It is clear, therefore, that:

When the ratio of dilution for the special dilutions is the same as that in the actual test, Equations (27) are rigidly correct and independent of any error in the titration of the salt solution.

We may go further, and say:

When the difference between t_2 and t'_2 is sufficiently small, the error in t will have no appreciable effect on the values of the ratios computed by Equations (27), and it is also apparent that any errors due to the inequality of t_2 and t'_2 will be negligible under similar conditions.

Now, to apply Equations (27) to the problem of Section 15, we must suppose a special dilution made up by taking, say, 10 grammes of salt solution, diluting this to 1 000 grammes with normal head-water and then diluting 10 grammes of the resultant dilution to, say, 600 grammes, so as to produce a dilution of 1 to 6 000, or $R' = 6\,000$. Breaking up a dilution in this way keeps down the size of the quantities treated in the laboratory.*

Under this treatment, a sample of 500 grammes of the dilution would, after evaporation to about 10 grammes, titrate to 50.335 c.c. of silver nitrate. This evaporation to 10 grammes requires that the sample of dilute salt solution should also be of 10 grammes weight, though a dilution of 10 grammes to 1 kg. would cause the titration to be one ten-thousandth of the titration for 1 kg., as shown in Table 2, Section 15, which is 45.62 c.c., or $t = 45.62$. All this

* In making successive dilutions, however, it is always better to make the partial dilutions equal, wherever practicable, this being on the side of economy and accuracy. Thus, if the two flasks had equal capacities of 750 c.c., the ratio would have been $75^2 = 5\,625$, instead of 6 000. The latter ratio is used merely for the purpose of a particular illustration.

requires that $\rho = 100$ and $\rho_2 = 50$, so that $\rho\,\rho_2 = 5\,000$. If we neglect the error in $e_1 \div e_2$, calling this ratio unity, we may write

$$T = \rho\,\rho_2\,t\,\frac{e}{e_2} = 5\,000 \times 45.62 = 228\,100 \ldots\ldots\ldots\ldots (34)$$

Simultaneously with the evaporations of the special dilution, there would be evaporated a number of tail-water samples which, in accord with the problem of Section 15, titrate to 55.10 c.c. of silver nitrate, or $t_2 = 55.10$.

The value of y then becomes

$$y = \frac{55.10 - 50.335}{228\,100 - 50.335} = 0.00002089 \ldots\ldots\ldots\ldots (35)$$

Now, it can be shown that the errors due to titration, under the circumstances of the problem, would not exceed 0.01 c.c. in the titration of t'_2, and not more than 0.03 c.c. in the titration of t, in order to leave the same relative error in all three titrations. See Section 43.

Indeed, these assumed errors are certainly large rather than small, as will appear in the study of errors, Section 42 *et seq.* Hence, affecting the titrations, t, t'_2, with these errors, which are to be subtracted in both cases, we shall have for the new value of y, observing that now $e \div e_2$ is properly made equal to unity,

$$\text{corrected } y = \frac{55.10 - 50.325}{227\,950 - 50.0} = 0.00002095 \ldots\ldots\ldots\ldots (36)$$

By the last of Equations (27) the ratio of dilution in the tail-race will be, for Equations (35) and (36), respectively,

$$\left.\begin{aligned} F_{35} &= \frac{6\,000}{1 + 5\,999\,y_{35}} = 5\,332 \\ F_{36} &= \frac{6\,000}{1 + 5\,999\,y_{36}} = 5\,330 \end{aligned}\right\} \ldots\ldots\ldots\ldots (37)$$

Thus, by neglecting the errors of titration, when $t = 45.62$, $t_2 = 55.10$, $t'_2 = 50.325$, the method of special dilutions gives the discharge correct to 2 in 5 332, or with an error of less than 0.04%, because, by applying the corrections as in Equation (36), the true ratio of dilution is seen to be 5 330.

Now, in all actual cases, t, t_2, and t'_2 can, without much trouble, be brought into much closer agreement than has been here supposed, and by a second treatment they may even be made equal if desired. Hence, it is clear that the laboratory work can be made to yield results which

are precise to one or two hundredths of 1%, in so far as systematic errors are concerned. As to accidental errors, there is no difficulty in checking the means of two sets of six or eight titrations to within 0.04 or 0.05%, so that a precision of $\frac{1}{10}$ of 1% is not difficult to realize.

If Equation (30) and the last of Equations (37) were actual cases, instead of mere illustrative problems, it would be wise to consider the latter as the more exact, or as having the greater weight, because errors in t, that is, in the numerator of Equation (30) and in the denominators of x and y, have much less effect on Equations (27) than on Equations (14).

Although the problems of this and the preceding section have been given as merely illustrative, the suppositions are based on the figures of Tables 2, 4, 5, and 6, which are derived from the results of actual tests. The errors thus computed, therefore, are very close estimates of any actual errors that would occur under similar circumstances.

17.—Ratio of Evaporation Nominal Only.—The ratio of evaporation has been defined and introduced into Equations (31) and (33). It should be observed that this is merely a nominal ratio of evaporation, as the evaporation is not a measured quantity. Its real significance in the equations may be better understood as being the ratio of the weight of a sample, as measured for evaporation, to the weight of a sample of dilute salt solution, as measured for titration. Thus, S is the weight of a dilute salt solution sample, say 10 grammes, and $\rho_2 S_2$ is the true weight of a tail-water sample, about 500 grammes, S_2 being equal to S for samples of equal volume. Thus $\rho_2 = 50$ in that case.

The evaporations will be closely enough equal if they are all made as nearly equal to the volume of a dilute salt solution sample as the eye can estimate by actual comparison when both samples are in exactly equal casseroles. A dilute salt solution sample can be used as a "check" for the purpose of this comparison.

18.—Volumetric Equations Involving Densities and Strengths.—Equations (27) contemplate the measurement of samples by weight, and determine the weight of water passing through the turbine during a test. It is more common in chemistry to measure quantities of liquid samples by volume. This will add somewhat to the complexity of the

equations, as has been explained. However, the equations will prove convenient for many purposes, and are therefore set forth, as follows:

According to the definitions, Section 7, we must have

$$\left.\begin{aligned} f &= \frac{W_1}{w} = \frac{Q_1 d_1}{q d} = r \frac{d_1}{d} \\ F &= \frac{W_2}{w} = \frac{Q_2 d_2}{q d} = R \frac{d_2}{d} \\ f' &= \frac{W'_1}{w'} = \frac{Q'_1 d_1}{q' d} = r' \frac{d_1}{d} \\ F' &= \frac{W'_2}{w'} = \frac{Q'_2 d'_2}{q' d} = R' \frac{d'_2}{d} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots (38)$$

Hence, in order to introduce volumes and densities in Equations (27), Section 14,* it will be necessary merely to substitute these values of the weight ratios in the proper places and thus obtain

$$\left.\begin{aligned} r &= \frac{r'}{1 + \left(r' \frac{d_1}{d} + 1\right) x} \qquad x = \frac{p_2 - p'_2}{p - p_2} \\ r &= \frac{R' \frac{d'_2}{d_1} - \frac{d}{d_1}}{1 + R' \frac{d'_2}{d} x} \\ R &= \frac{\left(r' \frac{d_1}{d} + 1\right)}{1 + r' \frac{d_1}{d} y} \qquad y = \frac{p_2 - p'_2}{p - p'_2} \\ R\dagger &= \frac{R' \frac{d'_2}{d_2}}{1 + \left(R' \frac{d'_2}{d} - 1\right) y} \end{aligned}\right\} \quad \ldots (39)$$

By the method of special dilutions, the equations involving densities and strengths.

19.—Volumetric Equations Involving Concentrations and Strengths.—These equations may be deduced by substituting $c \div p$ for d, etc., in Equations (39), but the following method is easier and serves better

* These substitutions may be made in other equations. For example, in Equations (14), Section 11, to obtain equations involving r and R.

† It is remarked that this last equation is really the only one which is entirely free from the properties of the normal head-water. Also notice that concentrations, densities, and percentages are connected by Table 1, Equations 2, 3, 4, and Plates XXXIII, XXXIV, and XXXV, so that, any one being given, the other two may be found. Thus these equations express the volumetric ratios in terms of either density or percentage when taken in conjunction with the tables, equations, or diagrams.

to familiarize one with the relations existing among the quantities involved. It is first to be remembered that Equations (39) were established by the use of Equations (14), Section 11, Equations (20-21), Section 13, Equations (27), Section 14, and Equations (38), Section 18. Further, let

W_{s1} = the total weight of salt in the head-water;
W_{s2} = " " " " " " " " tail-water;
w_s = " " " " " " " " salt solution;
$f_s = W_{s1} \div w_s$;
$F_s = W_{s2} \div w_s$;
P = ratio of total weight of salt solution to weight of salt content;
P_1 = ratio of total weight of head-water to weight of salt content;
P_2 = ratio of total weight of tail-water to weight of salt content.

Thus
$$P = 1 \div p, \text{ etc.}$$

Then
$$\left.\begin{aligned} W_{s1} P_1 + w_s P &= W_{s2} P_2 \\ W_{s1} + w_s &= W_{s2} \\ \text{or} \quad f_s P_1 + P &= F_s P_2 \\ \text{and} \quad f_s + 1 &= F_s \end{aligned}\right\}$$

from which
$$\left.\begin{aligned} f_s &= \frac{P - P_2}{P_2 - P_1} \\ F_s &= \frac{P - P_1}{P_2 - P_1} \end{aligned}\right\} \text{(See Equations (14))} \quad \ldots\ldots\ldots\ldots (40)$$

Hence, if we put
$$\left.\begin{aligned} X &= \frac{P_2 - P'_2}{P - P_2} \\ Y &= \frac{P_2 - P'_2}{P - P'_2} \end{aligned}\right\} \text{(See Equations (20))} \quad \ldots\ldots\ldots (41)$$

and, similar to Equations (38),
$$\left.\begin{aligned} f_s &= \frac{W_{s1}}{w_s} = \frac{Q_1 c_1}{q c} = r \frac{c_1}{c} \\ F_s &= \frac{W_{s2}}{w} = \frac{Q_2 c_2}{q c} = R \frac{c_2}{c} \end{aligned}\right\} \text{(See Equations (38))} \quad \ldots\ldots (42)$$

We are, by analogy, able to write out at once, from a mere inspection of Equations (27),

$$f_s = \frac{f'_s}{1 + (f'_s + 1)\,X}$$

$$f_s = \frac{F'_s - 1}{1 + F'_s\,X}$$

$$F_s = \frac{f'_s + 1}{1 + f'_s\,Y}$$

$$F_s = \frac{F'_s}{1 + (F'_s - 1)\,Y}$$

(See Equations (27)) ..... (43)

By further analogy, from Equations (39), substituting c for d, etc.,

$$r = \frac{r'}{1 + \left(r'\frac{c_1}{c} + 1\right)\frac{p}{p'_2}\,x}$$

$$r = \frac{R'\frac{c'_2}{c_1} - \frac{c}{c_1}}{1 + R'\frac{c'_2}{c}\,\frac{p}{p'_2}\,x}$$

$$R = \frac{r'\frac{c_1}{c} + 1}{1 + r'\frac{c_1}{c}\,\frac{p}{p_2}\,y}$$

$$R = \frac{R'\frac{c'_2}{c_2}}{1 + \left(R'\frac{c'_2}{c} - 1\right)\frac{p}{p_2}\,y}$$

$X = \frac{p}{p'_2}\,x$

$Y = \frac{p}{p_2}\,y$

By the method of special dilutions, the equations involving concentrations and strengths.

(See Equations (39)) ..... (44)

20.—It must not be overlooked that in some cases it may be of more advantage to compute the strength, or other property, of the normal head-water, from the special dilutions than to eliminate it entirely from the equations. Thus, for example, by substituting in Equations (28) from Equations (38), there may be obtained

$$p_1 = \frac{R'\frac{d'_2}{d}\,p'_2 - p}{r'\frac{d_1}{d}}$$

or

$$p_1 = \frac{R'\frac{d'_2}{d}\,p'_2 - p}{R'\frac{d'_2}{d} - 1}$$

(See Equations (28)) By the method of special dilutions, values of p_1 to check constancy of salt content in head-water. ..... (45)

$$r = \frac{d}{d_1}\,\frac{p - p_2}{p_2 - p_1} \qquad \left.\begin{aligned} f &= r\frac{d_1}{d} \\ F &= R\frac{d_2}{d} \end{aligned}\right\} \qquad (46)$$

$$R = \frac{d}{d_2}\,\frac{p - p_1}{p_2 - p_1}$$

which, by giving the value of p_1, d_1, etc., may serve as a check on the constancy of head-water concentration, density, etc.

Observe that the first of Equations (46) may be used to find the volume of a quantity of solution with which a known volume of standard solution has been mixed, and the second equation gives the volume resulting from the mixture of two solutions when the strengths and densities of all the solutions and the volume of one of them are known, thus

$$\left.\begin{aligned} Q_1 &= rq \\ Q_2 &= Rq \end{aligned}\right\} \qquad (47)$$

21.—Similarity of Relations Involving Density, Strength, and Concentration.—In view of the definitions, there may be written, along with Equation (11),

$$\left.\begin{aligned} W_1 p_1 + w p &= W_2 p_2 \\ Q_1 d_1 + q d &= Q_2 d_2 \\ Q_1 c_1 + q c &= Q_2 c_2 \end{aligned}\right\} \qquad (48)$$

By dividing by w or q, there results

$$\left.\begin{aligned} f p_1 + p &= F p_2 \\ r d_1 + d &= R d_2 \\ r c_1 + c &= R c_2 \end{aligned}\right\} \qquad (49)$$

By a further division by p, d, or c, using the following notation

$$\left.\begin{aligned} P_x &= \frac{p_x}{p} \\ D_x &= \frac{d_x}{d} \\ C_x &= \frac{c_x}{c} \end{aligned}\right\} \qquad (50)$$

there results

$$\left.\begin{aligned} fP_1 + 1 &= FP_2 \\ rD_1 + 1 &= RD_2 \\ rC_1 + 1 &= RC_2 \end{aligned}\right\} \qquad (51)$$

Moreover, from Equation (6),

$$C = PD, \text{ etc.} \qquad (52)$$

To these should be added the analogous forms, Equations (40), and the other relations of Section 19. The similarity is apparent, and will be of frequent service in deriving other relations.

22.—Volumetric Equations Involving Densities and Concentrations.—The first two of these equations may be derived most simply by solving the last pair of Equations (51) for r and R, respectively.

Hence,

$$\left.\begin{aligned} r &= \frac{D_2 - C_2}{D_1 C_2 - D_2 C_1} \\ R &= \frac{D_1 - C_1}{D_1 C_2 - D_2 C_1} \end{aligned}\right\} \qquad (53)$$

Of course, to solve for quantities, we must use

$$\left.\begin{aligned} Q_1 &= qr \\ Q_2 &= qR \end{aligned}\right\} \qquad (54)$$

These equations are subject to the objection that the titration for c_i will be more or less erroneous, as set forth fully in Sections 15 and 16. Hence C_i and D_i, also, are likely to be in error by an amount larger than the permissible margin. This source of error may be removed by resort to special dilutions, also discussed in the two sections mentioned. An independent solution could be made by forming, from Equations (53), two equations corresponding to Equations (18), Section 13, after which the solution would proceed in a manner quite similar to that given in the remainder of Section 13; but the solution may be made more expeditiously by simply substituting in Equations (39), as follows:

$$\left.\begin{aligned} x &= \frac{p_2 - p'_2}{p - p_2} = \frac{\frac{c_2}{d_2} - \frac{c'_2}{d'_2}}{\frac{c}{d} - \frac{c_2}{d_2}} = \frac{C_2 D'_2 - C'_2 D_2}{D_2 - C_2} \frac{1}{D'_2} \\ y &= \frac{p_2 - p'_2}{p - p'_2} = \frac{\frac{c_2}{d_2} - \frac{c'_2}{d'_2}}{\frac{c}{d} - \frac{c'_2}{d'_2}} = \frac{C_2 D'_2 - C'_2 D_2}{D'_2 - C'_2} \frac{1}{D_2} \end{aligned}\right\} \qquad (55)$$

Thus, with the aid of the second of Equations (51), noting that $d_1 \div d = D_1$, etc.,

$$\left.\begin{aligned} r &= \frac{r'}{1 + (r' D_1 + 1) x} \\ r &= \frac{R' \frac{D'_2}{D_1} - \frac{1}{D_1}}{1 + R' D'_2 x} \\ R &= \frac{r' D_1 + 1}{1 + r' D_1 y} \\ R &= \frac{R' \frac{D'_2}{D_2}}{1 + (R' D'_2 - 1) y} \end{aligned}\right\} \quad \left\{\begin{array}{l} \text{By the method of special} \\ \text{dilutions, the equations} \\ \text{involving densities and} \\ \text{concentrations. See} \\ \text{Equations (55).} \end{array}\right\} \quad \ldots (56)$$

the last equation being the only one of the set free of D_1 as well as C_1.

The values for x and y are here supposed to be the last of those given in Equations (55), thus Equations (56) will be expressed entirely in terms of densities and concentrations. Of course, the reader will understand that the densities should be known, in fact, rather than be computed from the relations given in Table 1, Plates XXXIII, XXXIV, or XXXV, or Equations (4), which are supposed to apply only to solutions of pure sodium chloride in distilled water rather than to solutions of commercial salt in river water. However, in many cases, results will be sufficiently exact under the latter circumstances, where there is not much foreign matter carried in the water.

In case it is desired to check the salt content of the normal head-water from day to day, or test to test, as in Equations (28), we may, from the special dilutions, determine the value of C_1 in Equations (53) and then determine r or R by the same equations, but making use of the titrations of the tail-water instead of those of the special dilutions, just as in Equations (28); thus, from Equations (53),

$$\left.\begin{aligned} \frac{D_2 - C_2}{r} &= \frac{D_1 - C_1}{R} \\ \text{or} \qquad C_1 &= \frac{r D_1 - R D_2 + R C_2}{r} \\ &= \frac{R C_2 - 1}{r}, \end{aligned}\right\} \quad \ldots (57)$$

since $rD_1 - RD_2 = -1$, *by* Equations (51).

Hence, with special dilutions,

$$C_1 = \frac{R'\,C'_2 - 1}{r'}$$

$$r' = \frac{D'_2 - C'_2}{D_1\,C'_2 - D'_2\,C_1}$$

$$r = \frac{D_2 - C_2}{D_1\,C_2 - D_2\,C_1}$$

or $$R = \frac{D_1 - C_1}{D_1\,C_2 - D_2\,C_1}$$

(Equations involving densities and concentrations. Values of C_1 to check constancy of salt content.) ...(57)

These equations may be obtained directly from Equations (28) by making the proper substitutions.

23.—Volumetric Shrinkage Coefficient.—It was explained in Section 5 that a shrinkage of volume takes place when two salt solutions of different densities are mixed. In many cases this shrinkage is relatively small, but there are cases where it would have to be taken into account when errors of less than 1% are of consequence, as, for example, where equal volumes of strong salt solution and distilled water are mixed.

It will appear presently that the total shrinkage of volume is only a relatively small fraction of the volume of the stronger of the two solutions which are mixed. Hence it will always be convenient to consider that the resultant volume is equal to the volume of the weaker solution plus a certain fraction of the original volume of the stronger solution.

Thus, if q is the volume of the stronger solution and Q_1 that of the weaker, the resultant volume may conveniently be written

$$Q_2 = Q_1 + kq \quad \ldots\ldots(58)$$

where k is a positive fraction which will be found to approximate unity more or less closely in any case. It will be shown that the minimum value which k can have is approximately 0.96.

If Equation (58) is divided by q the result will be

$$R = r + k \quad \ldots\ldots(59)$$

which is an equation of fundamental importance.

Now, from Equation (48),

$$Q_2 = Q_1 \frac{d_1}{d_2} + q \frac{d}{d_2}$$

$$= Q_1 + \left(Q_1 \frac{d_1}{d_2} - Q_1\right) + q \frac{d}{d_2}.$$

Therefore

$$\left.\begin{aligned} Q_1 + k\,q &= Q_1 + \left[\left(r\,\frac{d_1}{d_2} - r\right) + \frac{d}{d_2}\right]q \\ \text{or} \qquad k &= r\left(\frac{d_1}{d_2} - 1\right) + \frac{d}{d_2} \end{aligned}\right\} \quad \ldots\ldots(60)$$

which gives the value of k in terms of the densities of the two solutions and their resultant mixture.

By analogy

$$k = r\left(\frac{c_1}{c_2} - 1\right) + \frac{c}{c_2} \quad \ldots\ldots(61)$$

By taking the last two of Equations (49) with Equation (59), we have

$$\left.\begin{aligned} R &= r + k \\ Rd_2 &= rd_1 + d \\ Rc_2 &= rc_1 + c \end{aligned}\right\} \quad \text{or} \quad \left\{\begin{aligned} R &= r + k \\ RD_2 &= rD_1 + 1 \\ RC_2 &= rC_1 + 1 \end{aligned}\right\} \quad \ldots\ldots(62)$$

from which, by eliminating R and r, we may derive a value for k in terms of densities and concentrations, thus

$$k = \frac{C_2 - C_1 - (D_2 - D_1)}{C_2 D_1 - C_1 D_2} \quad \ldots\ldots(63)$$

which may also be easily obtained by noting from above that $k = R - r$, and then taking the difference between the two Equations (53).

Therefore k may be computed by any one of Equations (60), (61), or (63). In making the calculations, it is necessary, of course, to make use of the physical properties of the salt solution, as given in Table 1 or Plates XXXIII, XXXIV, and XXXV. It must be remembered that Equations (1), (2), (3), and (4) are limited to very dilute solutions.

By applying Equation (60) for various ratios of mixture, r, at different temperatures, Table 3 has been computed for two strengths of salt solution and for ratios of mixture from 1 to infinity. The correctness of the figures depends on the accuracy of Landolt and Börnstein's tables. If the table is in error, or if the salt solution contains impurities, the figures will include a corresponding error,

though this error, in many cases, will not be large. It may clarify matters to reiterate that the table has been computed for the water of the St. Lawrence River as it reached the power-house during the turbine tests considered herein. The tabular values of k, therefore, will not exactly check the values obtained in Section 5.

TABLE 3.—VOLUMETRIC SHRINKAGE COEFFICIENT FOR THE WATERS OF THE ST. LAWRENCE RIVER DURING THE TURBINE TESTS CONSIDERED IN THIS PAPER.

(*A*) { Concentration of salt solution = 300 grammes per liter.
Concentration of canal water = 0.014 gramme per liter.

Temperature, in degrees, centigrade.	Values of r.							
	1	5	10	50	100	1 000	10 000	Infinity.
0°	0.9877	0.9748	0.9715	0.9672	0.9667	0.9663	0.9662	0.9662
10°	0.9898	0.9796	0.9771	0.9730	0.9725	0.9721	0.9720	0.9720
15°	0.9913	0.9818	0.9790	0.9745	0.9740	0.9736	0.9736	0.9735
20°	0.9916	0.9834	0.9821	0.9760	0.9755	0.9751	0.9751	0.9751
40°	0.9934	0.9862	0.9834	0.9804	0.9800	0.9796	0.9796	0.9796

(*B*) { Concentration of salt solution = 275 grammes per liter.
Concentration of canal water = 0.014 gramme per liter.

0°	0.9891	0.9777	0.9753	0.9715	0.9710	0.9706	0.9706	0.9706
10°	0.9913	0.9820	0.9798	0.9764	0.9760	0.9757	0.9756	0.9756
15°	0.9924	0.9842	0.9817	0.9777	0.9773	0.9770	0.9769	0.9769
20°	0.9927	0.9858	0.9836	0.9790	0.9786	0.9782	0.9782	0.9782
40°	0.9941	0.9887	0.9862	0.9829	0.9826	0.9822	0.9822	0.9821

NOTE.—The figure in the fourth decimal place is likely to be in error.

The relative shrinkage of volume on mixing the two salt solutions, as computed in Section 5, is given by the equation:

$$\frac{\Delta Q}{Q} = \frac{1 - k}{r + 1},$$

where ΔQ is the shrinkage of the mixture from the sum, Q, of the volumes of the two components before being mixed.

In computing the values of p_2, it will be of advantage to notice that

$$p_2 = \frac{r c_1 + c}{r d_1 + d} \qquad (64)$$

which may be derived from Equations (8), Section 9, by putting $w = q\,d$, $p = c \div d$, etc.

To facilitate interpolations for values of k, Fig. 1 has been prepared; its use will be understood without further explanation.

COEFFICIENT OF VOLUMETRIC SHRINKAGE
FOR
MIXTURES OF SALT SOLUTION AND CANAL WATER.

The coefficient is the fraction by which the volume of the stronger solution must be multiplied to obtain the volume which when added to the volume of the weaker gives the volume of the subsequent mixture.

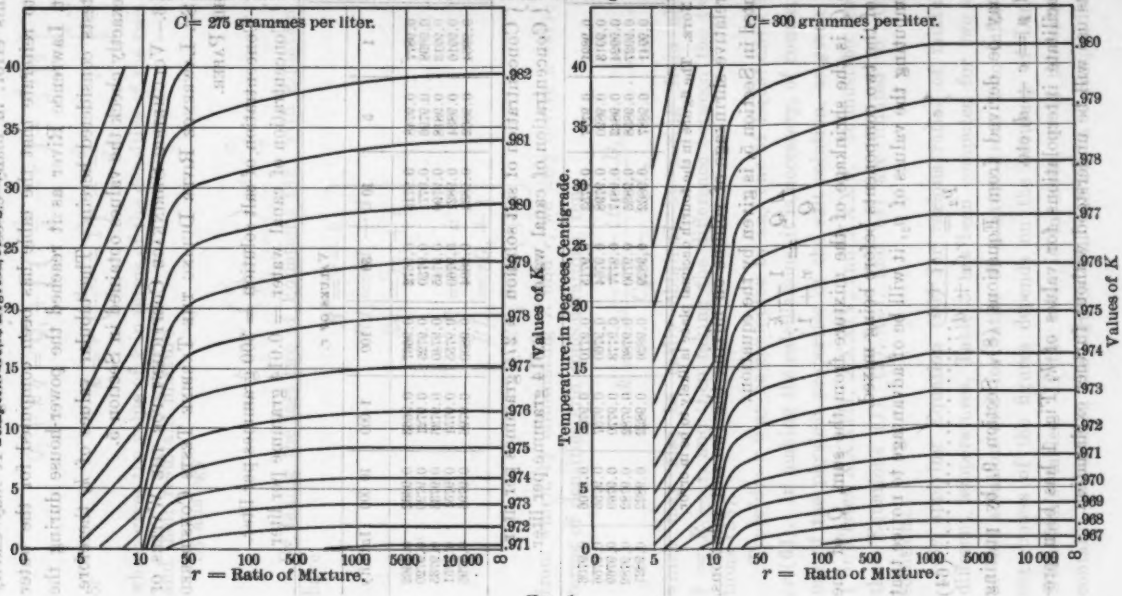


FIG. 1.

24.—Minimum Coefficient of Shrinkage.—It was stated in Section 23 that the least value of k is approximately 0.96. It will be seen from an inspection of Table 3 that the coefficient decreases when the density of the strong salt solution increases, when r increases, and when the temperature falls, the lowest tabular value being about 0.966. If we limit the temperature to zero, and take the strongest salt solution at that temperature, we may find, by Equations (60), (61), or (63), the value of k for any ratio of mixture. The least value will be for an infinite ratio, thus the problem is to find the limit of k when r is made to approach infinity as a limit.

Take, for example, Equation (63), noting that

$$C_1 = P_1 D_1, \text{ etc. (see Equations (6) and (50))} \quad \ldots\ldots(65)$$

$$D_1 = D_0 + n \frac{c}{d^2} P_1, \text{ etc. (see Equations (1) and (50))} \quad \ldots\ldots(66)$$

$$\begin{aligned} C_2 - C_1 &= P_2 D_2 - P_1 D_1 \text{ (from Equation 66)} \quad \ldots\ldots(67) \\ &= (P_2 - P_1)\left[D_0 + n \frac{c}{d^2}(P_2 + P_1)\right] \\ &= (P_2 - P_1)(D_2 + D_1 - D_0) \end{aligned}$$

$$D_2 - D_1 = n \frac{c}{d^2}(P_2 - P_1) \text{ (Equation (66))} \quad \ldots\ldots(68)$$

$$C_2 D_1 - C_1 D_2 = (P_2 - P_1) D_2 D_1 \text{ (Equation (65))} \quad \ldots\ldots(69)$$

Hence, substituting these values of $(C_2 - C_1)$, $(D_2 - D_1)$, and $(C_2 D_1 - C_1 D_2)$, there may be written, cancelling the common factor $(P_2 - P_1)$,

$$\left.\begin{aligned} k &= \frac{D_1 + D_2 - D_0 - n\frac{c}{d^2}}{D_2 D_1} \text{ (value of } n \text{ in Equations (1) and (2)).} \\ \text{or } k &= \frac{(d_1 + d_2 - d_0)\, d - n\, c}{d_1 d_2} \end{aligned}\right\} \quad \ldots\ldots(70)$$

where n depends on the temperature, as in Equations (1) and (2).

If now, the ratios of mixture and dilution are made to approach infinity as a limit, the value of d_2 will change, approaching d_1 as a limit, while c and n are supposed to remain constant. Therefore, the limit of k is

$$\text{Lt. } k]_{r=\infty} = \frac{(2 d_1 - d_0)\, d - n\, c}{d_1^2} \left\{\begin{array}{l}\text{(salt solution, canal} \\ \text{water, zero degrees.)}\end{array}\right\} \quad \ldots\ldots(71)$$

It will be observed, also, that k will decrease when there is a decrease in the salt content of the water with which the salt solution is

mixed. Hence, to get the least value of the limit of k under the conditions supposed, let the mixture be of the strongest salt solution at 0° cent., with distilled water. That is, $d_1 = d_0$. Therefore

$$\text{Limit } k]_{r=\infty} = \frac{d_0\, d - n\, c}{d_0^2} \left\{ \begin{array}{l} \text{(salt solution, distilled} \\ \text{water, zero degrees.)} \end{array} \right\} \quad \ldots(72)$$

for the case supposed.

On reference to Tables 1 and 2 for numerical values, there may be found for a saturated solution of salt, of 26.4% at 0° cent.,

$$\left. \begin{array}{r} n = 0.772 \\ c = 319.15 \\ d_0 = 0.99987 \\ d = 1.2089 \end{array} \right\} \quad \ldots\ldots\ldots\ldots(73)$$

from which the least value of k is found to be

$$\left. \begin{array}{l} \text{Least value of } k \text{ for a mixture of} \\ \text{saturated salt solution at 0° cent.,} \\ \text{with distilled water at the same} \\ \text{temperature, } r = \infty. \end{array} \right\} \quad 0.9626 \ldots\ldots\ldots\ldots(74)$$

From the fifth column of Table 1 it will be found that the volume of distilled water required to make a liter of saturated salt solution at 0° cent. is 889.87. Hence, if Equations (2), Section 3, as derived from Landolt and Börnstein's tables are correct to the limit, it must follow that a molecule of salt always displaces a portion of the water in which it is dissolved, however dilute the solution may be. If the curves of Plate XXXIII approach a slope of 45° or more, when the concentration approaches zero, which does not appear to be the case, this statement would not be true.

Otherwise the limit of k, in Equation (74), would necessarily be 0.88987, the volume of distilled water in a cubic centimeter of saturated salt solution at 0° cent.

Of course, the same limit could be calculated from either Equations (60) or (61).

An actual observation of the value of k was made during the calibration of the solution tank by the chemical method on November 23d, 1914. (See Tables 34 and 35.) The tank was first filled with canal water to the point marked A on the calibration scale of the gauge-board, shown at A on Plate XXXVII. A calibrator full of strong

salt solution containing about 265 grammes of salt per liter was then discharged into the tank and a thorough mixture was secured by running the circulating pump. The level of the mixed solution in the tank was then found to be at the point, *B*, on the gauge-board. At the left of Plate XXXVII there is a full-sized sketch of the calibration scale.

Now, each division on the scale corresponds to the volumetric delivery of the calibrator. Thus, the space of 1.51 in. from Graduation 12 to Graduation 13 represents the volume delivered by the calibrator at this part of the scale. The rise of level due to the addition of a calibrator of salt solution, observed after a thorough mixture with the canal water originally in the tank, was 1.47 in., as shown on Plate XXXVII. Therefore, the observed value of k in this experiment is $147 \div 151 = 0.973$.

By referring to Table 3, Section 23, and Fig. 1, the value of k, for a ratio of 65 and temperature of 5° cent., which are approximate data from the test, is seen to compare with the observed value above, as follows:

Observed value of k, November 23d, 1914.....0.973
Interpolated tabular value for conditions of test...0.976+

which is within the limit of error probable in reading the elevation of the surface of the solution with the calibration scale. It is not safe, however, to conclude from this one observation that a molecule of salt always displaces a portion of the water in which it is dissolved. The conditions of the test, too, are far removed from the limiting conditions supposed to exist when deriving the limit in Equation (74).

25.—Successive Dilutions.—It has been seen that it is very serviceable, and conducive to accuracy, to dilute the salt solution so as to lead to titrations of samples of the same concentration. It is sometimes convenient to dilute the tail-water to a certain degree of concentration for the same purpose. Hence, it is important to have the relations of the quantities under such circumstances expressed in algebraic form.

From Equations (62), Section 23, we have, for a first dilution, symbolized by R', r', c'_2, etc.,

$$c' = \frac{r' c_1 + c}{R'} \qquad (75)$$

and for a second dilution of this tail-water with a new portion of normal head-water, symbolized by R'', r'', etc.

$$c'' = \frac{r'' c_1 + c'}{R''} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (76)$$

Substituting the value of c' from Equation (75), we derive

$$c'' = \frac{r'' c_1 + \dfrac{r' c_1 + c}{R'}}{R''} = \frac{R' r'' c_1 + r' c_1 + c}{R' R''}$$

Therefore,

$$c'' = \frac{(R' r'' + r') c_1 + c}{R' R''} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (77)$$

Thus, the whole operation results in a solution which is exactly the same as though the ratio of mixture of normal head-water and salt solution had been $R' r'' + r'$ and the ratio of dilution $R' R''$. Denoting the ratio of mixture and ratio of dilution of the resulting solution by r and R, respectively, the statement may be expressed as follows:

$$\left.\begin{aligned} r &= R' r'' + r' \\ \text{and} \quad R &= R' R'' \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (78)$$

We may, of course, also write

$$r = (r' + k')\, r'' + r' = r' r'' + k' r'' + r' \quad \ldots\ldots\ldots\ldots\ldots\ldots (79)$$

and

$$\begin{aligned} r + k &= (r' + k')\,(r'' + k'') \\ &= r' r'' + k' r'' + r' k'' + k' k'' \quad \ldots\ldots\ldots\ldots\ldots\ldots (80) \end{aligned}$$

We may also deduce relations between k, k', and k''. Thus

$$\left.\begin{aligned} k &= R - r \\ &= R' R'' - R' r'' - r' \\ &= R' k'' - r' \\ &= (r' + k')\, k'' - r' \\ &= k' k'' + r' k'' - r' \\ &= k' k'' + r' (k'' - 1) \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (81)$$

26.—It will aid the comprehension considerably to deduce the relations of Equations (78) by direct reasoning rather than by formulas. Take, for example, a volume, x, of salt solution and dilute with a volume r' times larger, that is, with a volume, $r'x$. Then, by our theory and definitions, the resulting mixture will occupy a volume $k'x + r'x = (r' + k')x$.

Hence 1 liter of the mixture contains $x \div (r' + k')x = 1 \div (r' + k')$ of original salt solution, and $r'x \div (r' + k')x = r' \div$

$(r' + k')$ of normal head-water, the resulting volume of this liter, of course, being

$$\frac{k'}{r' + k'} + \frac{r'}{r' + k'} = 1 \text{ liter.}$$

If, now, a second dilution is made by taking a volume, y, of the first mixture and diluting with a volume r'' times larger, that is, with a volume, $r''y$, we shall have taken the equivalents of

$$\frac{y}{r' + k'} \text{ of original salt solution}$$

and $$\frac{y\,r'}{r' + k'} \text{ of normal head-water}$$

to mix with $r''y$ of normal head-water.

The resulting mixture, therefore, is the same as though

$$\frac{y}{r' + k'} \text{ of original salt solution and } \frac{y\,r'}{r' + k'} + r''\,y$$

of normal head-water had been mixed in one operation.

Therefore,

$$r = \frac{\left(\frac{r'}{r' + k'} + r''\right) y}{\frac{y}{r' + k'}} = (r' + k')\,r'' + r'$$

or

$$r = R'\,r'' + r'$$

the same as in Equations (78), which was proved by algebraic substitutions.

27.—Successive Dilutions in Laboratory.—Suppose now, that, for convenience in laboratory work, the tail-water is diluted in such a manner that r'' liters of normal head-water are mixed with 1 liter of tail-water before taking samples for evaporation. That is, the ratio of mixture is r'' and the ratio of dilution is $r'' + k'' = R''$. Then, if the diluted tail-water is evaporated in the usual manner and the test is reduced as though this had been the actual tail-water, the ratios of mixture and dilution for the actual tail-water can be easily computed from the results, thus,

$$\left.\begin{aligned} r' &= r - R'\,r'' \\ r' &= r - \frac{R}{R''}\,r'' \end{aligned}\right\} \text{(See Equations (78))} \quad \ldots\ldots\ldots (82)$$

which latter is in terms of the laboratory dilution and final ratios of mixture and dilution computed from the modified laboratory procedure indicated previously.

In a similar manner, we may have, directly from Equations (79),

$$\left.\begin{aligned} r' &= \frac{r - k'\,r''}{1 + r''} \\ R' = r' + k' &= \frac{r - k'\,r''}{1 + r''} + k' \\ &= \frac{r}{1 + r''} + \frac{k'}{1 + r''} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots (83)$$

but such forms are not entirely independent of the first dilution, as they involve k', or other functions of r' or k'.

Again, from Equations (78),

$$R' = \frac{R}{R''} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (84)$$

which may be used to compute R' when R has been computed from the results of titrations of the samples of diluted tail-water and R'' has been observed. In terms of these same quantities, r' may be expressed as follows, from Equations (82),

$$\left.\begin{aligned} r' &= r - \frac{R}{R''}\,(R'' - k'') \\ &= r - R + \frac{R}{R''}\,k'' \\ &= \frac{R}{R''}\,k'' - k \\ &= \frac{R\,k'' - k\,R''}{R''} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots (85)$$

28.—Inverse Dilutions.—Considerable space has been devoted to the theory of mixtures of salt solutions. The study has been necessary to educate the writer sufficiently to enable him to offer very simple proofs of the fundamental volumetric equations depending on ratios of mixture and dilution only. Before proceeding with the discussion it will be well, however, to recall the relations existing between the tail-water and special dilution samples which have been prepared in the laboratory.

The tail-water and special dilution are both mixtures of salt solution with normal head-water, the respective ratios of mixture being r and r' and the respective ratios of dilution R and R'.

As it is the object of the special dilution to reproduce a solution as nearly as possible like the tail-water, we shall always have, more or less closely,

$$R \text{ approximates } R'$$

and

$$r \text{ approximates } r'$$

If it happens that the special dilution is stronger in salt than the tail-water, then $R > R'$ and $r > r'$; but, if the contrary is true, $R' > R$ and $r' > r$. In the former case salt solution may be added to the tail-water until the mixture is just like the special dilution, and in the latter case salt solution may be added to the special dilution until it is just like the tail-water. In each case the quantity of salt solution necessary will depend entirely on the values of R and R', or of r and r', and the quantity of the tail-water, or special dilution, with which it is mixed. In either case the necessary ratio of dilution of salt solution will be indicated by R''.

Such a dilution of salt solution by another solution would convert the latter into a stronger one, whereas, a dilution of the solution with normal head-water would convert it into a weaker one. In this sense the dilution of salt solution with tail-water, or special dilution, will be called an inverse dilution.

29.—Volumetric Equations Involving Ratios Only.—Suppose, first, that the tail-water is weaker in salt than the special dilution, or $R > R'$ and $r > r'$. Let R'' be the ratio of dilution necessary to convert tail-water into special dilution by adding salt solution. Take sufficient tail-water to make the resulting mixture have a volume, R', and therefore contain r' of normal head-water. This must be $\frac{r'}{r} R$ of tail-water, which necessarily contains $\frac{r'}{r}$ of salt solution, as R of tail-water contains r of normal head-water and the resultant mixture must contain only r'. It is also apparent from the premises that the requisite quantity of salt solution to mix with the tail-water is $R' \div R''$ and that the total quantity of salt solution in the resulting mixture must be unity. Therefore,

$$\frac{R'}{R''} + \frac{r'}{r} = 1$$

from which

$$r = \frac{r'}{1 - \frac{R'}{R''}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (86)$$

30.—The tail-water still being weaker than the special dilution, let r'' be the ratio of mixture necessary to convert the former into the latter by adding salt solution. Take a volume, R, of tail-water, which contains r of normal head-water and 1 of salt solution. It will require $R \div r''$ of salt solution to reduce the mixture to the condition of special dilution, as tail-water is being mixed with the salt solution and r'' is the ratio of mixture. Thus, the resulting mixture contains $1 + \frac{R}{r''}$ of salt solution and r of normal head-water. The resulting mixture, however, is exactly the same as the special dilution. Therefore,

$$\frac{r}{1 + \frac{R}{r''}} = r' \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (87)$$

31.—From Equations (86) and (87) it is clear that

$$\left(1 + \frac{R}{r''}\right)\left(1 - \frac{R'}{R''}\right) = 1 \quad \ldots\ldots\ldots\ldots\ldots\ldots (88)$$

and that the quantity of salt solution required to reduce a solution the ratio of mixture of which with the same salt solution is r to a stronger one having a ratio of r' is such that

$$\left.\begin{aligned} R'' &= \frac{R'}{1 - \frac{r'}{r}} \\ \text{and} \quad r'' &= \frac{R}{\frac{r}{r'} - 1} \end{aligned}\right\} \quad (r > r') \quad \ldots\ldots\ldots\ldots (89)$$

32.—Reciprocal Dilutions.—It will be seen by reference to Section 25 that the second dilution with normal head-water there discussed converts special dilution into tail-water; and the dilution with salt solution examined in Sections 29 and 30 returns the mixture to its original condition as regards its relative composition. A pair of dilutions thus related may be called a pair of reciprocal dilutions.

It is clear from Section 25, Equations (78), that $R'r''$ is the quantity of normal head-water necessary to convert R' of special dilution to the condition of tail-water, and, from Section 30, that $R \div r''$ of salt solution will return the mixture to its original condition of special dilution in which the ratio of mixture is r'. Therefore the quantity, $R'r''$, of normal head-water and the quantity, $R \div r''$, of salt solution must be mixed in the same ratio as the special dilution, or

$$\left.\begin{aligned} \frac{R'\,r''}{\frac{R}{r''}} &= r' \\ \text{and} \qquad r''\,r'' &= R''\,r' \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (90)$$

From Section 29, if $\frac{r'}{r}R$ of tail-water is mixed with $\frac{R'}{R''}$ of salt solution, the mixture will be of volume, R', will contain a unit volume of original salt solution, and r' of normal head-water. Consequently, the $\frac{r'}{r}R$ of tail-water must have contained r' of normal head-water and $1 - \frac{R'}{R''}$ of salt solution. Therefore, using Equations (78),

$$\frac{r - R'\,r''}{1 - \frac{R'}{R''}} = r$$

from which

$$R''\,r'' = r \qquad (r > r') \ldots\ldots\ldots\ldots\ldots (91)$$

Expanding the product in Equation (88), there results

$$\frac{R}{r''} - \frac{R'}{R''} - \frac{R'\,R}{R''\,r''} = 0 \ldots\ldots\ldots\ldots\ldots (92)$$

Hence,

$$\left.\begin{aligned} \frac{R}{r''} &= \frac{\frac{R'}{R''}}{1 - \frac{R'}{R''}} \qquad (r > r') \\ \frac{R'}{R''} &= \frac{\frac{R}{r''}}{1 + \frac{R}{r''}} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots (93)$$

which may also be written

$$\left.\begin{aligned} \frac{R}{r''} &= \frac{1}{\frac{R''}{R'} - 1} \\ \frac{R'}{R''} &= \frac{1}{\frac{r''}{R} + 1} \end{aligned}\right\} \quad (r > r') \qquad \ldots\ldots\ldots\ldots\ldots\ldots (94)$$

From Equations (90) we have, as $R'' - r'' = k''$,

$$k'' = \frac{R'\,r - R\,r'}{r - r'} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (95)$$

which reduces easily to

$$k'' = \frac{k'\,r - k\,r'}{r - r'} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (96)$$

As the greater r must be associated with the smaller k (see Table 3 or Fig. 1), it must follow that

$$k'' > k \text{ and } k'' > k'.$$

When r and r' are nearly equal, and relatively large, it is clear that $k'' = k = k'$, practically. Indeed, the limit of k'' for fixed temperatures is $k' - r' \frac{d\,k'}{d\,r'}$, when r approaches r' as a limit. Fig. 1 shows that $r \frac{d\,k}{d\,r}$ is small, relatively to k, for all likely values of r.

33.—Since the discussion of Section 25, it has been assumed generally that the tail-water was a weaker solution than the special dilution, that is, that $r > r'$. If the contrary is true, and $r' > r$, then it would be necessary to dilute the tail-water with normal head-water to convert it to a solution like the special dilution, and to introduce salt solution into the special dilution to reproduce the conditions of tail-water. The foregoing equations for the successive and inverse dilutions would still hold, all the reasoning applying exactly as before, provided that r' and r, and R' and R are mutually interchanged in the equations, while the definitions of R'', r'', R'' and r'' would require a corresponding modification agreeable to the changed sense of the operations.

34.—*Volumetric Equations Involving Concentrations Only.*—From Equations (62),

$$\left.\begin{aligned} R &= r + k \\ R\,C_2 &= r\,C_1 + 1 \end{aligned}\right\} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (97)$$

from which,

$$\left.\begin{aligned} r &= \frac{1 - k C_2}{C_2 - C_1} \\ R &= \frac{1 - k C_1}{C_2 - C_1} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (98)$$

in terms of concentrations which are relative with respect to the concentration of the strong salt solution.

In terms of the actual concentrations,

$$\left.\begin{aligned} r &= \frac{c - k c_2}{c_2 - c_1} \\ R &= \frac{c - k c_1}{c_2 - c_1} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (99)$$

which are derived from Equations (98) by multiplication by c. Thus

$$c_1 = c\, C_1.$$

These forms, however, are subject to the errors of titration which affect c_1 relatively to c and c_2, as has been explained in Sections 15 and 16. In order to overcome this objection, the method of special dilutions is available, as described in the sections referred to and elsewhere in this paper.

35.—The preceding discussions enable us to write out at once the equations pertinent to the method of special dilutions and involving only the concentrations and one coefficient of shrinkage. These equations are the most convenient where the samples are measured volumetrically, but, as has been explained in Section 10, the equations of Sections 13 and 14 are much better adapted to the requirements of turbine testing. Resuming the assumption that $r > r'$, the definitions of Sections 29 and 30 require that

$$\left.\begin{aligned} r'' &= \frac{c - k'' c'_2}{c'_2 - c_2} \\ R'' &= \frac{c - k'' c_2}{c'_2 - c_2} \end{aligned}\right\} \quad (r > r') \quad \ldots\ldots\ldots\ldots\ldots\ldots (100)$$

$$0.96 < k'' < 1$$

which are the same in form as Equations (99).

But, if the assumption is that $r' > r$, we must have

$$\left.\begin{aligned} r'' &= \frac{c - k'' c_2}{c_2 - c'_2} \\ R'' &= \frac{c - k'' c'_2}{c_2 - c'_2} \end{aligned}\right\} \quad (r' > r) \quad \ldots\ldots\ldots\ldots\ldots\ldots (101)$$

Hence, changing the assumption is equivalent to substituting $-r''$ for R'' and $-R''$ for r'' in Equations (100) or (101).

Now, under the assumption that $r > r'$, and consequently that $c'_2 > c_2$ we have, by Equations (86) and (100),

$$\left.\begin{aligned} r &= \frac{r'}{1 - \frac{R'}{R''}} \\ \text{or} \qquad & \qquad (c'_2 > c_2) \\ r &= \frac{r'}{1 + R' \frac{c_2 - c'_2}{c - k'' c_2}} \end{aligned}\right\} \ldots\ldots\ldots\ldots (102)$$

But, under the assumption that $r' > r$, and consequently that $c_2 > c'_2$, we have, interchanging r and r', and putting R' for R, by Equations (87) and (101),

$$\left.\begin{aligned} r &= \frac{r'}{1 + \frac{R'}{r''}} \\ \text{or} \qquad & \qquad (c_2 > c'_2) \\ r &= \frac{r'}{1 + R' \frac{c_2 - c'_2}{c - k'' c_2}} \end{aligned}\right\} \ldots\ldots\ldots\ldots (103)$$

We have, by Equations (93),

$$R = \frac{\frac{R'}{1 + \frac{k''}{r''}}}{1 - \frac{R'}{R''}}$$

which reduces easily to

$$\left.\begin{aligned} R &= \frac{R'}{1 + \frac{k'' - R'}{r''}} \\ \text{or} \qquad & \qquad (c'_2 > c_2) \\ R &= \frac{R'}{1 - (k'' - R') \frac{c_2 - c'_2}{c - k'' c'_2}} \quad \text{(see Equations (100))} \end{aligned}\right\} \ldots (104)$$

But when $r' > r$, we have, by substituting r for r', R for R', and *vice versa*, by Equations (93) and 101),

$$R = \frac{\frac{R''}{r''} R'}{1 + \frac{R'}{r''}}$$

or

$$R = \frac{R'}{1 - (k'' - R') \frac{c_2 - c'_2}{c - k'' c'_2}} \qquad (c_2 > c'_2) \qquad \text{.....(105)}$$

Therefore, it is proved generally, whether $c_2 > c'_2$, or the contrary, that the general equations for r and R by the method of special dilutions are

$$r = \frac{r'}{1 + (r' + k') \frac{c_2 - c'_2}{c - k'' c_2}}$$

$$r = \frac{R' - k'}{1 + R' \frac{c_2 - c'_2}{c - k'' c_2}} \qquad \text{(General formulas, method of special dilutions.)}$$

$$r = \frac{r'}{1 + R' \frac{c_2 - c'_2}{c - k'' c_2}} \qquad (1 > k'' > 0.96) \qquad \text{....(106)}$$

$$R = \frac{R'}{1 + (R' - k'') \frac{c_2 - c'_2}{c - k'' c'_2}}$$

$$R = \frac{r' + k'}{1 + (r' + k' - k'') \frac{c_2 - c'_2}{c - k'' c'_2}}$$

36.—Equations (106) may, of course, be deduced in numerous other ways, for example, by making use of Equations (27) of Section 14, Equations (38) of Section 18, Equations (53) of Section 22, and the relations of Equations (100) and (101) of Section 35.

37.—Constancy of Salt Content in Head-Water.—Similar to Equations (28) of Section 14, and Equations (45) of Section 20, are the equations derived from either Equations (62) of Section 23, or Equations (98) and (99) of Section 34, when special dilutions are

made up in the laboratory, and the normal head-water titration is calculated instead of observed, thus,

$$\left.\begin{aligned} c_1 &= \frac{R'\,c'_2 - c}{r'} \quad \text{(Value } c_1 \text{ to check constancy of head-water content of salt.)} \\ r &= \frac{c - k\,c_2}{c_2 - c_1} \quad \text{(General formulas for method of special dilutions.)} \\ R &= \frac{c - k\,c_1}{c_2 - c_1} \quad (1 > k > 0.96) \end{aligned}\right\} \cdots\cdots (107)$$

The special dilutions eliminate the unbalanced evaporations for c_1, as compared with the evaporations for c_2.

Of course, it has not been forgotten that we may substitute v for c in these equations, so as to introduce the titrations per liter of the various solutions, instead of the concentrations. Indeed, by slight modifications of these formulas, the actual titrations, t, t_2, etc., may be substituted, as was illustrated in Equations (33) of Section 16, and as will be shown in the discussion of the theory involved in Equations (27), (28), (39), (44), (45), (106), (107), (108), and others of similar nature. This discussion appears in Section 45.

38.—Volumetric Equations Involving Densities Only.—Owing to the relations discussed in Section 21, there may be written, by analogy,

$$\left.\begin{aligned} r &= \frac{r'}{1 + R'\,\dfrac{d_2 - d'_2}{d - k''\,d_2}} \quad (1 > k'' > 0.96) \\ R &= \frac{R'}{1 + (R' - k'')\,\dfrac{d_2 - d'_2}{d - k''\,d'_2}} \end{aligned}\right\} \cdots (108)$$

which might be used for the purpose of determining volumes of discharge when facilities are at hand for determining densities with the requisite precision.

39.—Color.—Instead of d relating to the specific quantity of matter, or quantity of chemical, present, it may relate to the intensity of any other property, such, for example, as color, which can be diffused throughout the volume of the liquid, the term, d, being in that case a measure of the quantity of coloring matter necessary to give the liquid a measurable or comparable tint. In such case the tints due

to d_2 and d'_2 can be made to agree exactly, so that the second term of the denominator vanishes and we have simply

$$R = R' \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (109)$$

where R' is the only quantity necessary to measure, the colors of the tail-water and special dilutions being exactly matched. A colorimeter of special construction will evidently facilitate the determinations of R', and therefore of R.

40.—Electro-Metric Methods.—Time will not be taken up in entering into the great variety of methods which may be used in gauging by "dosing" the feed-water of a turbine. Suffice it to state that electro-metric methods are also being studied at the present time in various parts of the world, notably by Dr. Mellet and P. Dutoit, in Switzerland. The writer also has had made some tests of this character which seem promising.

Dr. Mellet's pamphlet, "Les Jaugeages par Titration Physico-Chemique," an extract from *Bulletin Technique* de la Suisse Romande, February 10th, 1915, page 25, gives the results of several gaugings by this method, with an account of the nature of the processes.

The writer's method of applying the conductivity test is: First, determine the conductivity of the tail-water; secondly, prepare a special dilution of the same salt solution which was used in the test with normal head-water, approximating as nearly as possible the actual ratio during the test; thirdly, with the same electrodes and the same electrical equipment used with the tail-water, determine whether the conductivity of the special dilution is greater, or less, than that of the tail-water; fourthly, measure from a burette the quantity of salt solution, or normal head-water, as the case may be, necessary to equate exactly the conductivity of the special dilution to that of the tail-water, the burette discharging directly into the special dilution, which is constantly stirred, until the electrical indicator shows that the two conductivities are equal; fifthly, the known ratio of dilution of the special dilution, determined from the measured quantities of the components, is then the required ratio of dilution for the test; sixthly, this ratio multiplied by the rate of discharge of the salt solution during the test gives the discharge of the turbine. Of course, the method of weights as against the method of volumes has the same advantages when measurements are made electrically as when they are made chemically.

PART II.

ERRORS.

Summary.

41.—Part II discusses at length the errors of the method and the procedure for their elimination. Corrections have been determined and tabulated for the method of unbalanced evaporations.

Incidentally, the method of balanced evaporations, or special dilutions, leads to an independent determination of the error produced by unbalanced evaporations, and proves that the writer's method of correcting the results by the latter method is correct.

A system of group titrations is devised, for the purpose of eliminating a large variety of errors.

The effect of non-uniformity of mixture of the chemical with the feed-water is shown to be less than many have heretofore supposed. This was shown in the tests by throwing the heavier concentration first to one side of the raceway and then to the other. The properly weighted concentrations give results which plot on a line, while the unweighted concentrations do not exhibit such discrepancies as would be a serious matter in tests of ordinary precision. In the best results, on Unit No. 13, the arithmetical mean concentrations would have served almost as well as the weighted mean.

The centrifugal pump is proved to be an excellent mixer of salt solution and canal water. Tests on such pumps can be executed with the utmost precision by the chemical method. Indeed, it is doubtful whether a better method can be found for the testing of pumps.

The weights for averaging the tail-water concentrations were determined by current meters in the tail-race. The still-water ratings, without correction, were used, but it is likely that some better method can be devised for determining weights.

The difficulties encountered in securing representative tail-water samples are related, with the means adopted for removing all uncertainty in this respect.

The method of designing and installing efficient sprinkler mixing systems is discussed, with the results of experiments bearing on sprinklers and perforations.

Pipette calibrations of an unusual character are given which show the combined and separate effects of temperature and concentration of

salt solution on the discharge of these instruments. It has been the practice, even by the United States Bureau of Standards, to calibrate a pipette at one temperature and then to assume that the changes in discharge due to changes in temperature are equal to the corresponding changes in volume of the pipette, thus disregarding the effects on the discharge of those properties of the contained liquids which change with the temperature.

It is shown herein that the change in discharge due to a change in temperature is entirely distinct from the change in volume of the pipette due to the same change in temperature, and that the combined effects of viscosity, surface tension, cohesion, and capillary action in general affect the discharge to a very much greater extent. The total change in discharge is several times the change in volume of the pipette. Isograms giving the discharge of pipettes for salt solutions of all concentrations at various temperatures are given as a ready means for determining the discharge.

The calibrations of flasks are given at all working temperatures, and rules are stated for effecting a perfect mixture by shaking the flasks containing mixtures of salt solutions and water. It does not appear that it is generally known how many times an ordinary liter flask must be inverted and shaken to secure a uniform mixture. A failure of the chemists to secure accurate mixtures during the early tests necessitated a special investigation to determine this question.

It is proved that there is no systematic error of any consequence in the chemical procedure developed and used during the tests. This conclusion is proved by the close agreement between the chemical and gravimetric calibrations of the salt solution tanks. The tests are of greater value on account of the unfavorable conditions under which they were executed than they would have been under the more propitious conditions of an extremely painstaking laboratory process.

It is shown that the wooden salt solution tank used during the tests was remarkably constant in volume, probably not changing its capacity by one-tenth of 1% during the entire season.

A method of eliminating the error introduced into the computed efficiency of a turbine by reason of an error in the density of the feed-water is given, and also a method for obviating the necessity for determining the density of the feed-water. That is, the density of feed-water has been entirely eliminated from the calculation.

The weight of the salt injected into the feed-water may be neglected in most cases.

The question is raised as to the correct method of determining the head acting on a turbine. It is shown that the velocity-head should be taken into account, and simple methods for its determination are discussed.

The effect on the tail-race concentration, and therefore on turbine efficiency, of variations of discharge and power during a test have been determined, and are shown to be negligible where the variations are less than certain limiting variations.

The relation of volume- to time-averages is discussed, so that, with a time-average concentration, a volume-average may be computed, and *vice versa*. Time-averages are shown to be accurate enough for most purposes. A device is suggested for determining volume-averages.

The sources of error in chemi-hydrometry may be classified as follows:

1.—*Initial Content of Chemical.*—There may be chemicals in the stream initially which react with the standard; or the dosing chemical may be affected by them.

2.—*Variability of Initial Content.*—The initial content of a chemical may be variable, and this variation may be appreciable from minute to minute.

3.—*Variability of Standard Chemical.*—The character of the standard, and consequently the equivalent weights of other chemicals, may change during a series of chemical analyses.

4.—*Indicator.*—The indicator used in volumetric analyses may act on the standard chemical. This is the case to an appreciable extent when chromate of potassium and silver nitrate are used.

5.—*Excess of Standard.*—An additional excess of standard chemical will usually be required to effect a definite determination of the end of the reaction.

6.—*Distribution of Chemical.*—The chemical is not likely to be distributed uniformly throughout the mass of the liquid to be measured, either as to time or place, except in the case of relatively small quantities of liquid.

7.—*Temperature Effects.*—The temperature during a test, or series of tests, and at the time of the ensuing treatment of samples in the laboratory, may be different at different times and places, thus introducing changes in the capacities of the measuring instruments, and corresponding changes in the volumes and properties of the various solutions used.

8.—*Changes in Calibrations.*—There may be other circumstances which cause changes in the calibrations of the various instruments, such for example as a change in the volume of the tank from which the dosing chemical is discharged and measured.

9.—*Time.*—Even if the watches have been carefully regulated and rated, there may be errors in observing, such, for example, as the personal error of an observer.

10.—*Density of Feed-Water.*—In turbine testing an error will be made if the density of the feed-water entering into the efficiency calculations is in error. This density may be determined or the error may be eliminated by the method of counter errors, explained in the sequel.

11.—*Weight of the Chemical.*—A similar error to that preceding will be made if the weight of the chemical injected into the head-water is not taken into account. However, in most cases this will be inappreciable.

12.—*Additions or Abstractions of Chemical.*—There may be additions or abstractions of chemical between the dosing and sampling stations, due either to natural causes, such as mineral springs, or to artificial causes, such as the operation of industries between the stations.

13.—*Additions or Abstractions of Water.*—There may be additions or abstractions of water between the stations due to natural or artificial causes.

14.—*Error in Tables.*—There may be errors in the physical constants used in the computations. For example, there may be errors in Table 1.

15.—*Errors in Theory.*—The theory may not be complete, or it may be fundamentally wrong. For example, it is shown later that there is an error in using Equations (17) of Section 11, without correcting v_1 for the error of titration, whereas,

Equations (27) and (28), Sections 13 and 14, are correct in this respect when the data are correctly observed.

16.—*Air in Feed-Water.*—On referring to Section 73, it will be seen that the density of the feed-water may be eliminated. It is also clear that the chemical method of measuring discharge does not include the volume, or weight, of any air, which may be entrained in the water temporarily as it passes the turbine.

42.—Net Error Due to Imperfect Theory.—In discussing the foregoing errors, it will be of advantage first to take up the net error due to imperfect theory, after which a detailed examination of the separate errors will be made.

In order to examine the theory, let

c_s = concentration of the standard chemical;

v = volume of the standard required to titrate a unit volume of sample;

V = volume of sample titrated;

e = equivalent of the dosing chemical, salt for example, per gramme of the standard chemical. In theory this will be the ratio of molecular weights, but in practice it will vary from such a ratio, due to various sources of error;

t = actual titration of a sample which is not exactly one unit in volume, thus $v = t \div V$, $v_2 = t_2 \div V_2$.

Then, for example, theoretically, we must have such equalities as

$$\left.\begin{aligned} c_2 &= \frac{t_2 c_s e}{V_2} \\ c &= k \frac{t_2 c_s e}{V_2} \end{aligned}\right\} \quad \text{or} \quad \left.\begin{aligned} Q &= q \frac{t_2 c_s e}{V_2} - \frac{t_1 c_s e}{V_1} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (110)$$

where e is strictly the ratio of chemical equivalents; but, in actual cases, it will seldom be found that such expressions constitute actual equalities; one side or the other will be the greater. Such a failure of theory may result from one or more sources of error.

It must not be supposed, therefore, that the ratio of chemical equivalents, e, deduced from the atomic weights of the elements involved, can be used with any degree of reliability in practical cases. Even where the chemicals are pure, there will be required a certain

excess of standard chemical in order to act on the indicator, and this excess will vary with circumstances. This is equivalent to saying that the practical value of e is variable to a slight extent.

It is necessary, therefore, to determine by actual observation what the net effect of the errors is, in any particular case, so that the observed discharge can be corrected; or, measures must be adopted so that the errors will be eliminated or reduced to inappreciable magnitudes.

43.—Practical Determination of Net Error of Titration for Equations (14), (17), (46), (53), (98), and (99).—Take Equations (99), Section 34, for example. These are virtually the same as Equations (53), Section 22, Equations (46), Section 20, or Equations (14) and (17), Section 11. If we substitute $Q_1 \div q$ for r, and $(Q_1 + kq) \div q$ for R, we have

$$Q_1 c_1 + qc = (Q_1 + kq)c_2 \ldots\ldots\ldots\ldots\ldots\ldots (111)$$

In terms of silver nitrate, or other chemical, this may be written

$$Q_1 v_1 + qv = (Q_1 + kq)v_2 \ldots\ldots\ldots\ldots\ldots\ldots (112)$$

v, v_1, and v_2 being the quantities of silver nitrate required per liter of the samples as used in the tests. Hence, if t, t_1, and t_2 are the actual quantities of silver nitrate required for the corresponding samples, we shall have

$$v = \frac{t}{q}, \quad v_1 = \frac{t_1}{Q_1}, \text{ and } v_2 = \frac{t_2}{Q_1 + kq} \ldots\ldots\ldots\ldots (113)$$

Substituting these values in Equation (112), we derive

$$t_1 + t = t_2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (114)$$

as might naturally be inferred without reference to Equation (112). That is to say, if three samples are prepared, consisting, respectively, of Q_1 of normal head-water, q of salt solution, and a mixture of Q_1 of normal head-water and q of salt solution, we should have the relation shown by Equation (114) between the titrations, because the last of the three samples is simply a mixture of samples exactly like the first two.

But, if the relation of Equation (114) fails to hold in actual trial, the fact will prove that the theory is incomplete, involving a systematic error, or that the methods do not eliminate the errors of titration. Moreover, if the samples, Q_1 and q, are of the same size as those

corresponding in the actual test, the difference between $(t_1 + t)$ and t_2 will determine the error due to the titration of samples of these sizes and concentrations.

A number of tests were made in this manner to determine whether the theory involved in the foregoing equations was exact or only approximate, with the result that the latter was shown to be the case. Table 4 exhibits the results which have been collected for this purpose. It must be impressed on the mind that these data are not derived from refined laboratory experiments, but were made in a rough field laboratory, hastily equipped in an old pumping station and storeroom, where there was more or less confusion resulting from the operation of large industrial plants in the immediate vicinity. Moreover, the chemical work was all done on a wholesale plan, under considerable pressure, so that no great attention could be paid to the details connected with any one operation. This was necessary in order to give capacity to the chemical laboratory, otherwise the large number of evaporations and titrations necessary could not have been executed.

On the other hand, it must be equally impressed on the mind that the best work done in this laboratory will undoubtedly rank with the best commercial laboratory practice. In fact, much that has been accomplished there will compare favorably with chemical research work of much greater pretension. It may be safely stated that the averages of half a dozen titrations as usually performed there will check the average of another similar set well within one-tenth of 1%, one-twentieth of 1% having been the error frequently observed. We have read much correspondence by eminent chemists on the subject of the accuracy of silver nitrate titrations for chlorine, and the accuracy obtained in our titrations will be better appreciated when it is stated that chemists as a rule do not rely on such titrations as being accurate to within 1 or 2 per cent. We have, however, to thank Dr. Mellet for his extended studies, which have resulted in a determination of the proper strengths of the solutions necessary to secure a degree of accuracy well within one-tenth of 1 per cent.

Nor is the degree of accuracy attained in the turbine tests to be inferred from the test titrations of Table 4, for most of these test titrations were crowded in at such times as the chemists chanced to be under less pressure of work than usual, and each series of test titrations, therefore, represents a break in the usual laboratory pro-

TABLE 4.—Errors of Titration Due to Unbalanced Evaporations.

Experimental Titrations in Chemical Laboratory.

(1)	(2)		(3)		(4)		(5)	(6)	(7)	(8)
Date, 1914.	Normal head-water.		Diluted salt solution.		Dosed tail-water.		N. H. + D. S. S. (Col. 2 + 3).	Diff. (Col. 5 — 4).	Error in percentage. (Col. 6 ÷ 4).	Remarks.
July 2 and 3..	.....	.....	53.50	53.60	78.30	78.55	.....	.....	.....	N. H. W. sample = 1 liter canal water.
	.....	.....	53.55	53.60	78.85	78.75	.....	.....	.....	Salt solution sample = 10 c.c. of solution.
	.....	.....	53.70	53.55	78.65	78.65	.....	.....	.....	Dosed tail-water sample = 1 liter of canal water + 10 c.c. of salt water solution.
	25.45	25.45	53.70	53.60	78.45	78.70	.....	.....	.....	$AgNO_3$ solution = 1.56 grammes per liter.
	25.60	25.55	53.40	53.35	79.05	79.05	.....	.....	.....	
	25.55	25.65	53.30	53.40	78.90	79.00	.....	.....	.....	
Totals.....	76.60	76.65	321.15	321.10	472.20	472.70	.....	.....	.....	
	153.25	.....	642.25	.....	944.90	.....	.....	.....	.....	
Ave.....	25.542	.....	53.521	.....	78.742	.....	79.063	+ 0.321	+ 0.408	
Temp.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	
July 6.....	25.75	25.70	53.90	54.05	78.95	79.00	.....	.....	.....	N. H. W. sample = 1 liter of canal water.
	25.60	25.57	54.05	54.10	79.10	79.15	.....	.....	.....	Salt solution sample = 10 c.c. of solution.
	25.68	25.68	53.80	54.00	79.30	79.40	.....	.....	.....	Dosed tail-water sample = 1 liter of canal water + 10 c.c. salt solution.
	25.75	25.75	53.95	53.95	79.05	79.10	.....	.....	.....	$AgNO_3$ solution = 1.56 grammes per liter.
	25.60	25.80	54.00	54.00	79.30	79.10	.....	.....	.....	
	25.70	25.55	54.10	54.05	79.30	79.33	.....	.....	.....	
Totals.....	154.08	153.70	323.80	324.15	475.00	475.08	.....	.....	.....	
	307.73	.....	647.95	.....	950.08	.....	.....	.....	.....	
Ave.....	25.644	.....	53.996	.....	79.173	.....	79.640	+ 0.467	+ 0.59	
Temp.....	20.5°C.	.....	.....	.....	20.5°C.	.....	.....	.....	.....	
July 8.....	25.55	25.55	54.00	54.00	79.45	79.45	.....	.....	.....	N. H. W. sample = 1 liter of canal water.
	25.50	25.55	54.00	53.95	79.40	79.40	.....	.....	.....	Salt solution sample = 10 c.c. of solution.
	25.55	25.80	54.00	54.00	79.35	79.40	.....	.....	.....	Dosed tail-water sample = 990 c.c. of canal water + 10 c.c. salt solution.
	25.80	25.70	54.10	53.95	79.55	79.43	.....	.....	.....	
	25.70	25.80	53.90	53.87	79.25	79.35	.....	.....	.....	
	25.90	25.80	53.85	53.85	79.35	79.20	.....	.....	.....	
Totals... .	154.00	154.20	323.85	323.62	476.35	476.23	.....	.....	.....	Add 1% N. H. W. titration to D. T. W. titration = 1 000 c.c canal water + 10 c.c. S.S.
	308.20	.....	647.47	.....	952.58	.....	.....	.....	.....	
Ave.	25.6833	.....	53.9558	.....	79.3817	.....	.....	.....	.....	$AgNO_3$ sol. = 1.56 grammes per liter.
Correction:					+ 0.2568					
Cor. Av.....	25.6833	.....	53.9558	.....	79.6385	.....	79.6391	+ 0.0006	+ 0.0	
Temp.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	

TABLE 4.—(*Continued.*)

(1)	(2)		(3)		(4)		(5)	(6)	(7)	(8)
Date, 1914.	Normal head-water.		Diluted salt solution.		Dosed tail-water.		N. H. + D. S. S. (Col. 2 + 3).	Diff. (Col. 5 − 4).	Error in percentage. (Col. 6 ÷ 4).	Remarks.
Averages:										
July 3.....	25.542	.....	53.521	.....	78.742	.....	.....	.....	.....	
" 6.....	25.644	.....	53.996	.....	79.173	.....	.....	.....	.....	
" 8.....	25.683	.....	53.956	.....	79.639	.....	.....	.....	.....	
Totals.....	76.869	.....	161.473	.....	237.554	.....	.....	.....	.....	
Ave.....	25.623	.....	53.824	.....	79.185	.....	79.447	+ 0.2629	+ 0.332	Average results.
Sample.....	*A*	*A*	*B*	*B*	*C*	*C*				Ratio of dilution of:
	55.51	55.40	27.73	27.59	13.95	13.85	.....	.....	.....(1)	Sample A / Sample B $= \frac{1\,002.96}{500.58} = 2.0036$
	55.49	55.42	27.75	27.80	13.84	13.82	.....	.....	.....(2)	Sample B / Sample C $= 2.0036$
Sept. 23.....	55.52	55.44	27.73	27.94	13.90	13.85	.....	.....	.....(3)	Sample A / Sample C $= 2.0036^2$
Totals.....	166.52	166.26	83.21	83.33	41.69	41.52	.....	.....	.....	Ratio of dilution takes flask and temperature correction into account.
	332.78	.....	166.54	.....	83.21	.....	.....	.....	.....	
Ave.....	55.463	.....	27.757	.....	13.868	.....	.....	.....	.....	$AgNO_3$ soln. = 1.56 grammes per liter.
Corr. Av.....	55.463	.....	55.514	.....	55.672	.....	.....	.....	.....	
Diff.....	− 0.209	.....	− 0.058	.....	.....	.....	.....	.....	.....	These are successive dilutions with distilled water beginning with Solution *A*.
Error %.....	− 0.375	.....	− 0.104	.....	.....	.....	Samples *A* and *B* referred to Sample *C*.			
Temp.....	.....	18.6°C.	.....	18.6°C.	.....	18.6°C.				
	13.20	13.20	27.85	27.75	40.95	40.95	.....	.....	.....	N. H. W. sample = 500 c.c. of canal water.
	13.15	13.20	27.85	27.80	40.90	40.90	.....	.....	.....	
	13.25	13.15	27.85	27.64	40.62	40.71	.....	.....	.....	Salt solution sample = 10 c.c. of solution.
August 7.....	13.10	13.18	27.75	27.75	40.80	40.70	.....	.....	.....	
	13.10	13.10	27.75	27.70	40.70	40.65	.....	.....	.....	Dosed tail-water sample = 500 c.c. canal water + 10 c.c. salt solution.
	13.05	13.05	27.75	27.70	40.70	40.65	.....	.....	.....	
	13.25	13.35	27.65	27.70	40.75	40.90	.....	.....	.....	
	13.33	13.45	27.90	27.70	40.83	40.85	.....	.....	.....	$AgNO_3$ solution = 1.56 grammes per liter.
	105.43	105.68	222.35	221.74	326.25	326.31	.....	.....	.....	
Totals.....	211.11	.....	444.09	.....	652.56	.....	.....	.....	.....	
Ave.....	13.194	.....	27.756	.....	40.785	.....	40.950	0.165	0.405	
Temp.....	25.5°C.	.....	26.0°C.	.....	25.5°C.	.....	.....	.....	.....	

TABLE 4.—(*Continued.*)

(1)	(2)		(3)		(4)		(5)	(6)	(7)	(8)
Date, 1914.	Normal headwater.		Diluted salt solution.		Dosed tail-water.		N. H. + D. S. S. (Col. 2 + 3).	Diff. (Col. 5 — 4).	Error in percentage (Col. 6 ÷ 4).	Remarks.
August 17	12.90	13.05	27.70	27.70	40.60	40.60				N. H. W. sample = 500 c.c. of canal water.
	13.15	13.05	27.70	27.70	40.95	40.60				Salt solution sample = 10 c.c. of solution.
	13.30	12.85	27.70	27.70	40.60	40.60				Dosed tail-water sample = 500 c.c. canal water + 10 c.c. salt solution.
	13.08	13.30	27.75	27.70	40.65	40.65				$AgNO_3$ solution = 1.56 grammes per liter.
	13.08	13.13	27.70	27.87	40.80	40.60				
	13.10	13.00	27.75	27.70	40.68	40.50				
	13.30	12.88	27.55	27.52	40.65	40.65				
	12.88	13.31	27.70	27.72	40.65	40.65				
Totals	104.79	104.57	221.55	221.61	325.58	324.85				
	209.36	.....	443.16	.....	650.43	.....				
Ave.	13.085	.....	27.697	.....	40.652	.....	40.782	0.130	0.320	
Temp.	22.5°C	.....	22.5°C.	.....	22.0°C.	.....				
August 25	13.30	12.98	55.63	55.55	40.53	40.31				N. H. W. sample = 500 c.c. of canal water.
	13.02	13.00	55.60	55.60	41.13	41.25				Salt solution sample = 10 c.c. of solution.
	13.00	13.00	55.60	55.60	41.35	40.35				Dosed tail-water sample = 500 c.c. canal water + 10 c.c. salt solution.
	13.15	13.08	55.62	55.62	40.13	41.35				
	13.18	12.96	55.60	55.61	33.90	40.35				
	13.20	13.00	55.68	55.60	47.80	40.76				
Totals	78.85	78.02	333.73	333.58	244.84	244.87				N. H. W. flask holds 500.5 c.c. D. T. W. flask ÷ 2 holds 501.5 c.c. Therefore, add 0.2% N. H. W. titration to D. T. W. titration for true value. $AgNO_3$ solution = 1.56 grammes per liter.
	156.87	.....	667.31	.....	489.21	.....				
Ave.	13.073	.....	27.805	.....	40.768	.....				
Correction:					+ 0.026					
Cor. Av.	13.073	.....	27.805	.....	40.794	.....	40.878	+ 0.084	0.205	
Temp.	20.7°C.	.....	21.5°C.	.....	20.7°C.	.....				
September 26 and 28	12.85	12.83	55.29	55.15	40.30	40.40				N. H. W. sample = 1 liter canal water and split by using ¼-liter flasks.
	12.90	12.90	55.10	55.25	39.90	40.87				Salt solution sample = 10 c.c. of solution.
	12.92	12.90	55.25	55.22	40.20	40.39				
	12.90	12.88	55.20	55.21	40.42	40.88				
Totals	51.57	51.51	220.84	220.83	160.82	162.54				Dosed tail-water sample = 1 liter canal water + 10 c.c. salt solution, split into two halves by ¼-liter flasks. Flasks were used so as to eliminate a correction. $AgNO_3$ solution = 1.56 grammes per liter.
	103.08	.....	441.07	.....	323.36	.....				
Ave.	12.885	.....	27.605	.....	40.420	.....	40.490	+ 0.070	+ 0.173	
Temp.	15.3°C.	.....	15.0°C.	.....	15.3°C.	.....				

TABLE 4.—(*Continued.*)

(1)	(2)		(3)		(4)		(5)	(6)	(7)	(8)
Date, 1914.	Normal head-water.		Diluted salt solution.		Dosed tail-water.		N. H. + D. S. S. (Col. 2 + 3).	Diff. (Col. 5 − 4).	Error in percentage. (Col. 6 ÷ 4).	Remarks.
Averages:										
August 7...	13.194	.....	27.756	.....	40.785	.....	.....	.....	.....	
" 17..	13.085	.....	27.697	.....	40.652	.....	.....	.....	.....	
" 25..	13.073	.....	27.805	.....	40.794	.....	.....	.....	.....	
September 26	12.885	.....	27.605	.....	40.420	.....	.....	.....	.....	
Totals.....	52.237	.....	110.863	.....	162.651	.....	.....	.....	.....	
Ave.....	13.059	.....	27.716	.....	40.663	.....	40.775	+ 0.112	+ 0.276	Average results.
					Mean per liter.....			+ 0.224	.....	.....
Sample.....	*A*	*A*	*B*	*B*	*C*	*C*	.....	.....	.....	Ratio of dilution of:
September 25	55.31	55.30	27.55	27.50	13.77	13.80	.....	.....	.....	Sample *A* / Sample *B* = 1 002.91 / 500.55 = 2.0036 (1)
	55.38	55.28	27.63	27.61	13.78	13.85	.....	.....	.....	Sample *B* / Sample *C* = 1 002.91 / 500.55 = 2.0036 (2)
	55.28	55.29	27.62	27.70	13.79	13.84	.....	.....	.....	Sample *A* / Sample *C* = 2.00362 (3)
Totals.....	165.92	165.87	82.80	82.81	41.34	41.49	.....	.....	.....	Ratio of dilution takes flask and temperature correction into account.
	331.79	.....	165.61	.....	82.83	.....	.....	.....	.....	$AgNO_3$ soln. = 1.56 gramme per liter.
Ave	55.298	.....	27.602	.....	13.805	.....	.....	.....	.....	
Cor. Av.....	55.298	.....	55.303	.....	55.420	.....	.....	.....	.....	
Diff.....	− 0.122	.....	− 0.117	.....	.....	.....	.....	.....	NOTE:	These are successive dilutions with distilled water beginning with Solution *A*.
Error %	− 0.222	.....	− 0.211	.....	.....	.....	.....	.....		
Temp.....	16.3°C.	.....	16.3°C.	.....	16.3°C.	.....	.....	.....		

cedure, and must be looked on as being conducted in a less systematic and efficient manner than the regular tests.

In studying the test titrations of Table 4 it will be best first to examine the work done in August, as that done in July was at a time when the chemists were less accustomed to their duties, and the facilities were not so well arranged as at a later time.

The tests of August 7th and 17th were arranged so that a sample of normal head-water consisted of a 500-c.c. flask filled to the graduation mark with this water. A sample of salt solution consisted of the quantity discharged from a 10-c.c. pipette, according to the directions set forth in the rules for the laboratory procedure, given in Section 88. A sample of dosed tail-water consisted of a 500-c.c. flask of normal head-water and a 10-c.c. pipette of salt solution mixed in a flask. It was scrupulously observed that the same flask and the same pipette were used throughout by any chemist in making up a set of three such samples. The salt solution was made of such a strength that the titration of this sample would run about half what it did in the usual test, and the normal head-water and dosed tail-water samples would run about as the usual ½-liter samples.

In the case of the tests of August 25th and September 26th to 28th, a somewhat different procedure was adopted. The salt solution was of about double the strength of that used on August 7th and 17th, and a sample of dosed tail-water was composed of approximately the half part of a mixture of 10 c.c. of salt solution and a liter of normal head-water. As before, only one flask and one pipette were used by any chemist in making up a set of samples. The exact division of the mixture of salt solution and dosed tail-water into ½-liter samples, was not attempted, thus leaving apparent inconsistencies among the records of the titrations of dosed tail-water samples, for these tests. The sum of the titrations of mates, however, is quite consistent from pair to pair, considering that no extra pains were taken during the work. In fact, the work was not as good as that done in the regular tests, as has been explained. Thus, the titrations of the samples on August 25th and September 26th to 28th were arranged to represent, as nearly as convenient, the actual titrations of a regular test.

As a consequence of the method of making up the samples and conducting the titrations, the sum of the normal head-water and salt

solution titrations, of any set, should, theoretically, be equal to the titration of the dosed tail-water sample of the same set. For example, on August 7th, we have 13.194 + 27.756 = 40.950, as compared with 40.785. Thus, the sum of the normal head-water and salt solution titrations exceeds that of the dosed tail-water by (40.950 — 40.785) = 0.165, an error of about 0.4% of the dosed tail-water titration.

This error is due to the fact that the concentration of the three samples is not the same. Indeed, the samples should be exactly alike as to all their characteristics, if the error of titration is to be eliminated. Dr. Mellet has accomplished the equation of the concentrations by proportioning sizes, dilutions, and evaporations in such a way that all samples will titrate alike. This, however, removes only a part of the difficulty, for the evaporations of the tail-water and normal head-water will necessarily be unbalanced, and the salt solution samples are dilutions with distilled water, instead of evaporations for concentration. As an example, it may be seen that it would have been necessary to evaporate about three times as much normal head-water as tail-water, in the tests now being discussed, in order to equate titrations, thus, perhaps, eliminating at least a part of the error detected above and observable in all the other tests.

Unfortunately, however, the equation of titrations requires unbalanced evaporations, and does not equate the errors of titration entirely, because there are other characteristics than concentration which must be equated simultaneously, as has already been pointed out in the theory which has been developed by the writer, in order to eliminate completely the errors of titration. There are organic and other substances in the normal head-water, which, when concentrated, operate in an entirely different manner to modify the titration of a sample of this solution as compared with a sample of either the dosed tail-water, brought to the same concentration, or the dilute salt solution sample, which contains no organic or other substances than the chemicals used in the test.

These differences are so marked that a little experience, even with such clear water as that of the St. Lawrence River, enables a person to know the class to which a particular sample belongs by mere inspection of its appearance. When evaporated in a casserole to a concentration of about 6 mg. in 10 c.c. of solution, the normal St. Lawrence

River water is reduced to a rusty muddy mixture, and the dosed tail-water sample, brought to the same concentration with only about one-quarter of the relative evaporation, is much cleaner and much more easily treated. The dilute salt solution of the same concentration, of course, contains no organic matter, and is, therefore, by far the clearest and best of all the samples.

This unbalancing of the evaporations, and the differences between concentrating by evaporation and diluting with distilled water modify to such an extent the ability to detect the end points of the three classes of titration that the errors of titration are not only still present, but may even be increased by completely equating the titrations. This remark applies to the method of concentrating the normal head-water sufficiently to equate the titrations, and the difficulty, it may be remarked, is the more serious where the normal head-water contains less chemical initially.

The differences between the tests of August 7th and 17th and the tests of August 25th and September 26th to 28th have been pointed out. It is not surprising, therefore, to note that there is a very observable effect on the relative errors of the end points. Thus, in the tests where 10 c.c. of dilute salt solution have titrated to only 27 or 28 c.c., the errors are 0.165 and 0.130, respectively, though the titrations of the tests where 10 c.c. of salt solution have titrated to about 55 or 56 c.c. are only 0.084 and 0.070 c.c., respectively.

An exhaustive research for the causes of these discrepancies might be made, but it is apparent that the waters of different streams may differ, and, moreover, the waters of the same stream may differ at different times. It is therefore much better to eliminate this error by adopting the special methods which will be fully explained presently. On this account, no extended study of this particular error will be made except to compute the average correction to be applied to the titration of the normal head-water to reduce it to a value which is in error by the same percentage as that which affects the tail-water titration. Incidentally, the approximate correction to be applied to the titration of the dilute salt solution when it differs from the titration of the tail-water can be ascertained.

In order to make the determination, it is desirable to use all the reliable tests which have been made. This, however, will not include

the tests of July, as they are to be looked on as being more or less untrustworthy, notwithstanding the fact that the average corrections computed from them will not differ greatly from those computed by using the other tests. It will be possible, however, to use the dilution tests at the bottom of the table, and thus secure advantages from all the tests which may be considered as entirely trustworthy. Probabilities might be formally applied, but the following treatment will be quite sufficient, in view of the paucity of refined observations.

If we represent by c_1, c, and c_2 the average of the corresponding titrations for August 7th and 17th, and let x and y be the corrections to c_1 and c, respectively, to be applied to these quantities to reduce them to quantities having the same percentage of error as c_2, supposing that the errors are functions of the magnitudes of the quantities, we should have,

$$(c_1 + x) + (c + y) = c_2 \ldots\ldots\ldots\ldots\ldots\ldots (115)$$

In similar manner, for the tests of August 25th and September 26th to 28th, observing that the normal head-water and tail-water samples titrate substantially the same as on August 7th and 17th, we should have for the average titrations

$$(c'_1 + x) + \frac{1}{2}(c' + z) = c'_2 \ldots\ldots\ldots\ldots\ldots\ldots (116)$$

For the mean titrations of the samples A and B of dilute salt solution at the bottom of Table 4, observing that, approximately, $A = c'$ and $B = c$, and that the ratio of dilution of A to get B is 2.0036 $= r$, say, we should have

$$\frac{1}{r}(A + z) = (B + y) \ldots\ldots\ldots\ldots\ldots\ldots (117)$$

Equations (115), (116), and (117) may now be written

$$\left.\begin{aligned} x + y &= c_2 - (c_1 + c) = -0.148 \\ x + \frac{1}{2}z &= c'_2 - (c'_1 + \frac{1}{2}c') = -0.090 \\ \frac{1}{r}z - y &= B - \frac{1}{r}A = +0.040 \end{aligned}\right\} \ldots\ldots\ldots (118)$$

which are easily computed from all the tests of Table 4, excepting those of July, which have been omitted for the reasons stated.

Thus:
$$\left.\begin{aligned} c_1 &= (13.194 + 13.085) \div 2 = 13.140 \\ c &= (27.756 + 27.697) \div 2 = 27.726 \\ c_2 &= (40.785 + 40.652) \div 2 = 40.718 \\ c'_1 &= (13.073 + 12.885) \div 2 = 12.979 \\ c' &= (55.610 + 55.210) \div 2 = 55.410 \\ c'_2 &= (40.768 + 40.420) \div 2 = 40.594 \\ A &= (55.298 + 55.463) \div 2 = 55.380 \\ B &= (27.602 + 27.757) \div 2 = 27.680 \end{aligned}\right\} \quad \ldots\ldots(119)$$

Apparently, we have three equations involving three unknowns, but it is observed that r is so nearly equal to 2 that small errors of observation introduce relatively large errors in the solutions. Thus, solving,

$$\left.\begin{aligned} x &= -\ 10.148 \\ y &= +\ 10.000 \\ z &= +\ 20.116 \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots(120)$$

If r is placed equal to 2 on the left of the last of Equations (118), which may be done without appreciable error, and $\frac{1}{2}z$ is eliminated from the last two equations, we obtain

$$x + y = -\ 0.130$$

as compared with

$$x + y = -\ 0.148$$

in the first equation. Thus, we have two approximate values of $x + y$, and the most probable value is the arithmetical mean or,

$$x + y = -\ 0.139.$$

By assuming, now, that $z = -\ y$, which cannot be very far from the truth, we shall have, omitting the last of Equations (118),

$$\left.\begin{aligned} x + y &= -0.139 \\ x - \tfrac{1}{2}y &= -0.090 \\ \text{therefore} \quad x &= -0.106 \\ \text{and} \quad y &= -0.033 \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots(121)$$

From the last of Equations (118) may be derived

$$z = -\ y = 0.027$$

a fair average value of y thus being about — 0.03.

The most important of these values is $x = -0.106$, which may be rounded off to -0.11, so that the correction per liter for titrations of 13 or 14 c.c. will be -0.22, and this figure must be deducted from the corresponding values of the doubled titrations, where the dosed tail-water titrates to about 40 c.c. Further, when the dilute salt solution titrates to about 55 and the tail-water to 40 c.c., the correction to be added to the titration of the salt solution is about 0.03 c.c. These values, of course, apply only to silver nitrate solutions of about the same concentrations as those used on the tests just discussed, which were about 1.5622 grammes per liter in all cases. The indicator, too, must be the same in all cases. However, the corrections would be only slightly different for concentrations of about 1.45 grammes per liter, so that no great changes need be made in the latter case.

TABLE 5.—SUMMARY OF AVERAGE CORRECTIONS FOR TITRATIONS WHERE $t_2 = 40$ C.C.

(APPLICABLE TO SUCH EQUATIONS AS (12), (14), (17), (19), (46), (53), AND (99).)

$AgNO_3 = 1.5622$ Grammes per Liter.

Indicator = 50 Grammes per Liter.

Titration, in cubic centimeters.	Corrections when tail-water titrates to about 40 c.c.
13.0	− 0.11
27.5	− 0.03
40.0	0.00
55.5	+ 0.03

From Table 5 we may compile, approximately, Table 6.

TABLE 5.—SUMMARY OF AVERAGE CORRECTIONS FOR TITRATIONS WHERE $t_2 = 55$ C.C.

(APPLICABLE TO SUCH EQUATIONS AS (12), (14), (17), (19), (46), (53), AND (99).)

$AgNO_3 = 1.5622$ Grammes per Liter.

Indicator = 50 Grammes per Liter.

Titration, in cubic centimeters.	Correction, $t_2 = 55$ c.c.
13.0	− 0.14
27.5	− 0.06
40.0	− 0.03
55.5	0.00

NOTE.—These corrections apply to ½-liter samples.

44.—Comparison of Theories.—Balanced and Unbalanced Evaporations.—Having shown how to correct the net error of titration in the case of the theory involved in Equations (12), (14), (17), (19), (46), (53), (98), and (99), it will be well to study a little more closely the theory involved in Equations (27), (28), (39), (44), (45), (56), (106), (107), and (108), with the view of eliminating this error entirely

The main feature which has been added to the theory in these equations is the very evident fact that titrations of samples which are just alike in all respects are in error by the same percentage when the titrations are performed with the same silver nitrate solution in the same way.

Take, for example, Equations (110) and (111), Sections 42 and 43, where

$$Q = q\,\frac{v\,c_s\,e - k\,v_2\,c_s\,e}{v_2\,c_s\,e - v_1\,c_s\,e} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (122)$$

supposing that all titrations have been reduced to the corresponding quantities of silver nitrate per liter of solution. We at once perceive that the new restriction imposes the requirement that e is not the same for any two titrations unless the two samples are exactly alike in all respects.

Thus, if t, t_1, and t_2 are the actual titrations of the sample volumes V, V_1, and V_2 which have been reduced from the original measured volumes of the samples by dilution or evaporation, and if ρ is the ratio of dilution of the salt solution and ρ_1 and ρ_2, respectively, the ratios of the volumes of original measured sample to the volumes of the residues after evaporation, which we may term "ratios of evaporation", then we may write Equation (122) as follows:

$$Q = q\,\frac{\frac{\rho\,t}{V}\,c_s\,e - k\,\frac{t_2}{\rho_2\,V_2}\,c_s\,e_2}{\frac{t_2}{\rho_2\,V_2}\,c_s\,e_2 - \frac{t_1}{\rho_1 V_1}\,c_s\,e_1} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (123)$$

in which the ratio of chemical equivalents is now considered to be different for different samples unless the actual samples of volumes V, V_1, and V_2 are exactly alike in all respects.

Now, it is easy enough to impose the condition that $t = t_1 = t_2$, by simply co-ordinating the values of ρ, ρ_1, ρ_2, V, V_1, and V_2, thus

$$\left.\begin{aligned} c &= \rho \frac{t}{V} c_s e \\ c_1 &= \frac{t_1}{\rho_1 V_1} c_s e_1 \\ c_2 &= \frac{t_2}{\rho_2 V_2} c_s e_2 \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (124)$$

from which, for $t = t_1 = t_2$, supposing that c, c_1, and c_2 are known,

$$\frac{c\,V}{\rho\,e} = \frac{c_1 \rho_1 V_1}{e_1} = \frac{c_2 \rho_2 V_2}{e_2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (125)$$

and, for the moment, supposing that $e = e_1 = e_2$ for equal titrations, and that S_1 and S_2 are the measured volumes which are respectively evaporated to V_1 and V_2,

$$\left.\begin{aligned} \frac{c}{c_1} &= \rho\,\rho_1 \frac{V_1}{V} = \rho \frac{S_1}{V} \\ \frac{c}{c_2} &= \rho\,\rho_2 \frac{V_2}{V} = \rho \frac{S_2}{V} \\ \frac{c_1}{c_2} &= \frac{\rho_2}{\rho_1} \frac{V_2}{V_1} = \frac{S_2}{S_1} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (126)$$

Thus, having decided on the value of ρ, the relative sizes of V, S_1, and S_2 become known, and the absolute values are known, if, in addition, the value of V is chosen. Now, as c_1 and c_2 cannot be equal, and as the actual volumes titrated must be the same, if the samples are to titrate equally and be of the same volume, we must have, for the moment, by our less restricted theory,

$$V = V_1 = V_2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (127)$$

if we are to have $e = e_1 = e_2$. Therefore

$$\frac{\rho_2}{\rho_1} = \frac{c_1}{c_2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (128)$$

which shows that the samples of normal head-water and tail-water must be concentrated by evaporation by very different quantities. Thus, if $c_1 = 10$ mg. per liter and $c_2 = 40$ mg. per liter, it is clear that we

shall have to evaporate four times as much normal head-water as tail-water to have equal titrations of equal volumes.

By our restricted theory, however, the samples of head-water and tail-water will not, by such a process, be alike in all respects, and therefore we shall have to deny that equal titrations of samples of equal volumes imply that $e = e_1 = e_2$. Hence, equations of the type of (14), (17), (46), (53), (98), (99), etc., do not admit of an elimination of the net error of titration, for the reasons just made apparent, except by making an independent study of the net corrections to be applied directly to titrations similar to that which has been previously made, Tables 5 and 6, Section 43.

45.—Elimination of the Error of Titration.—The error of titration being principally due to the dissimilarity of the conditions under which the head-water blank and the dosed tail-water samples are titrated, it is clear that this source of error may be avoided by eliminating the head-water titration and substituting for it the special dilution titration, which requires the use of such equations as (27), (28), (39), (44), (45), (56), (106), (107), and (108), and the preparation in the laboratory of a special sample as nearly like the tail-water sample as possible.

In order to have the "special dilution", as it will be called, exactly like the tail-water sample, it must be made from the same salt solution which has been used in the test; and the ratio of dilution of this salt solution with normal head-water must be as nearly as possible the same as that in the test. Moreover, the evaporation of these special samples must be conducted in exactly the same manner as the evaporation of tail-water samples, and to the same extent. This can easily be done by measuring out equal quantities of each and evaporating them over a water bath to the same extent.

It has already been explained, in Section 12, that it will always be possible to have sufficiently accurate advance knowledge of the value of the actual ratio of dilution, R, in the test, to enable a special dilution to be made in the laboratory substantially like the tail-water sample. It has also been explained, Section 17, that we need not measure accurately the ratio of evaporation of samples, merely making them alike, nominally, as small differences of concentration due to small differences in evaporation will not affect the titrations perceptibly.

Hence, this procedure is equivalent to the formal statements of the equations

$$\left.\begin{aligned} \rho_2 &= \rho'_2 \\ V &= V_2 = V'_2 \\ c &= \rho \frac{t}{V} c_s e \\ c_2 &= \frac{t_2}{\rho_2 V} c_s e_2 \\ c'_2 &= \frac{t_2}{\rho_2 V} c_s e'_2 \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (129)$$

It will be sufficient to discuss the nature of this procedure as it affects any one of the equations previously enumerated, for example, the first of Equations (106), Section 35, which is

$$r = \frac{r'}{1 + R' \dfrac{c_2 - c'_2}{c - k'' c_2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (130)$$

Substituting from Equations (129), and reducing somewhat,

$$r = \frac{r'}{1 + R' \dfrac{t_2 - t'_2}{\rho \rho_2 t \dfrac{e}{e_2} - k'' t_2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (131)$$

since, by our special laboratory process, e_2 and e'_2 are substantially equal.

Now,

$$T = \rho \rho_2 t \quad \ldots\ldots\ldots\ldots\ldots\ldots (132)$$

is the titration of a sample of salt solution of the same size as the measured tail-water, or special dilution, samples, as computed from the actual titration of a salt-solution sample of the same volume as the evaporated samples of tail-water or special dilution. That is, ρt is the titration of a sample of salt solution of the size of the evaporated samples, and ρ_2 is the ratio of the size of the measured sample to the size of the evaporated sample.

Thus, R' approximating R, we have,

$$r = \frac{r'}{1 + R' \dfrac{t_2 - t'_2}{T \dfrac{e}{e_2} - k'' t_2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (133)$$

in which T is the computed titration of a salt-solution sample of the same volume as that measured for the tail-water and special-dilution samples based on the titration of a dilute salt-solution sample of the same size as the nominal volume of the evaporated samples.

It may be readily seen that, when t_2 and t'_2 are nearly equal, an error in T has no appreciable effect on the value of r, and that when $t_2 = t'_2$, the result is entirely independent of T, and therefore of any error in T.

It may be observed that e and e_2 are not necessarily equal, as the sample of dilute salt solution, though of exactly the same size as the others, is different in other respects. It is a dilution with distilled water and, therefore, does not contain the same constituents as the others. This will, in a small degree, affect the end point determination; but it must be observed that the error is relatively small, under the conditions imposed, and affects only those terms of the equations which are themselves relatively small and tend to vanish altogether when the conditions of Equations (129) and (133) are approached. Therefore, it may be concluded that the special laboratory process outlined in the foregoing theory may be used to obviate the necessity for correcting titrations, as the chemical procedure involved eliminates the value of the concentration of the normal head-water from the equations, and with it the necessity of dealing directly with any of the properties of the head-water.

That the magnitude of any residual systematic error will not affect the result of a test appreciably, may be ascertained by discussions similar to those of Sections 16 and 43, where it is shown that the systematic error due to titrations of dissimilar samples in the turbine tests did not affect the results given by the method of special dilutions by perceptible quantities.

As Equations (106) and (107) express the same conditions, the error will be exactly the same in the two cases. Indeed, Equations (106) may be derived from Equations (107) by eliminating c_1 and solving for r, or R, as the case may be, although it is a solution where k'' has been inserted for a function of k', k, r', and r, which has a value nearly equal to unity. See Equation (96), Section 32.

Independently of Equations (106), it may be shown that an error in c tends to be eliminated in Equations (107). Suppose, for example,

that c becomes $c + \delta$ by reason of the error, δ. Then the resulting error in c_1 is

$$\frac{R' c_2 - (c + \delta)}{r'} - \frac{R' c_2 - c}{r'} = -\frac{\delta}{r'} \quad \ldots\ldots (134)$$

and the corresponding error in r is

$$\frac{(c + \delta) - k c_2}{c_2 - \left(c_1 - \frac{\delta}{r'}\right)} - \frac{c - k c_2}{c_2 - c_1} \quad \ldots\ldots (135)$$

which reduces to

$$\frac{\delta - \frac{\delta}{r'} \frac{c - k c_2}{c_2 - c_1}}{(c_2 - c_1) + \frac{\delta}{r'}} \quad \ldots\ldots (136)$$

When r and r', however, are closely equal, as is the case supposed in the use of Equations (107), we must have, closely,

$$r' = \frac{c - k c_2}{c_2 - c_1} \quad \ldots\ldots (137)$$

so that the error in r, expressed by Equation (136), tends to vanish when r and r' are closely equal.

46.—Errors Considered Separately.—Error 1.—Initial Content of Chemical.—It is clear that in eliminating c_1 by Equations (107), or similar equations, as mentioned in Section 37, and the special dilutions, the properties of the normal head-water are eliminated. This is the first source of error enumerated in Section 41. Equations (27), (28), (39), (44), (45), (56), (106), (107), and (108), result from the elimination according to the methods to be used in the test. By using any of these equations in connection with special dilutions, the error resulting from this source will be entirely eliminated, and, along with it, the necessity for titrating the blank sample of head-water. Indeed, it is possible to eliminate this error even where the initial content is variable, as will appear in the next section. In order to gain some idea of the nature of the normal head-water, as regards initial content of chemical, a study of Table 7, giving the values, in cubic centimeters, of silver nitrate during the entire series of tests, will be enlightening.

It will be observed from the figures in Table 7 that the content of chlorine in the St. Lawrence River was almost, if not quite, constant during the tests. The increase of the titration from about 26

c.c. of silver nitrate to 28 c.c. near the close of the tests is due to a change in the strength of the nitrate from a concentration of 1.5622 grammes per liter to 1.45 grammes dissolved in 1 liter of distilled water, which would correspond to a concentration of about 1.449 grammes per liter.

TABLE 7.—INITIAL CONTENT OF CHLORINE IN NORMAL HEAD-WATER DURING THE TURBINE TESTS, EXPRESSED IN HUNDREDTHS OF CUBIC CENTIMETERS OF SILVER NITRATE SOLUTION PER LITER OF SAMPLE, WHEN THE TITRATION IS OF ½ LITER EVAPORATED TO 10 C.C. IN A CASSEROLE OF ABOUT 11 CM. DIAMETER. THE STRENGTH OF THE SILVER NITRATE WAS APPROXIMATELY 1.56 GRAMMES PER LITER IN ALL THE TESTS EXCEPT IN THE SECOND SERIES ON UNIT NO. 11 WHEN IT WAS ABOUT 1.45 GRAMMES PER LITER. THE STRENGTH OF THE CHROMATE OF POTASSIUM INDICATOR WAS ABOUT 50 GRAMMES PER LITER IN ALL CASES, AND THE QUANTITY INTRODUCED INTO THE SAMPLE WAS FROM TWO TO THREE DROPS BY A "TK" DROPPING BOTTLE. THE GENERAL RULES FOR THE TITRATIONS ARE GIVEN IN SECTIONS 88 AND 89. THE FIGURES ARE CORRECTED TO AGREE WITH THE SECOND PART OF THIS TABLE, SEE SECTION 43, TABLES 5 AND 6.

Unit No. 11.		Unit No. 12.					
Test.	V_1, in $\frac{1}{100}$ c.c.	Test.	V_1, in $\frac{1}{100}$ c.c.	Test.	V_1, in $\frac{1}{100}$ c.c.	Test.	V_1, in $\frac{1}{100}$ c.c.
105	2 595	200	2 564	205	2 463	210	2 479
106	2 374	201	2 559	206	2 488	211	2 479
107	2 585	202	2 545	207	2 474	212	2 480
108	2 561	203	2 525	208	2 485		
109	2 590	204	2 475	209	2 484		
Unit No. 13.							
G	2 502	*W*	2 519	16	2 573	32–*A*	2 573
H	2 504	6	2 565	17	2 574	32–*B*	2 575
I	2 504	7	2 579	18	2 573	33	2 556
K	2 512	8	2 494	19	2 506	39	2 583
L	2 506	9	2 582	20	2 591	40	2 606
N	2 474	10	2 608	21	2 536	41	2 607
O	2 506	11	2 626	22–*A*	2 595	42	2 603
S	2 535	12	2 555	22–*B*	2 649	43	2 594
T	2 570	13	2 629	23–*A*	2 565	44	2 566
U	2 571	14	2 626	23–*B*	2 702	45–*A*	2 562
V	2 570	15	2 555	26–*A*	2 569	45–*B*	2 647
				26–*B*	2 616	46	2 594

TABLE 7 (*Continued*).—INITIAL CONTENT OF CHLORINE IN NORMAL HEAD-WATER DURING THE TURBINE TESTS, EXPRESSED IN HUNDREDTHS OF CUBIC CENTIMETERS OF SILVER NITRATE SOLUTION PER LITER OF SAMPLE, COMPUTED FROM THE TITRATIONS OF SPECIAL DILUTIONS AND SALT SOLUTIONS. THE STRENGTH OF THE NITRATE AND CHROMATE OF POTASSIUM INDICATOR WAS THE SAME IN ALL CASES AS THOSE INDICATED IN THE FIRST PART OF THIS TABLE.

COMPUTING VALUES OF V_1 FROM SPECIAL DILUTIONS.

UNIT NO. 11.

Test.	V_1, in $\frac{1}{100}$ c.c.	Test.	V_1, in $\frac{1}{100}$ c.c.	Test.	V_1, in $\frac{1}{100}$ c.c.	Test.	V_1, in $\frac{1}{100}$ c.c.
105	2 560	111	2 747	116	2 775	121-*A*	2 813
106	2 575	112	2 823	117	2 719	122-*A*	2 751
107	2 564	113	2 775	118	2 816	*120-*B*	3 116
108	2 618	114	2 822	119	2 802	121-*B*	2 798
109	2 637	115	2 782	120-*A*	2 825	122-*B*	2 807

UNIT NO. 12.

200	2 534	204	2 477	208	2 512	212	2 460
201	2 561	205	2 467	209	2 474		
202	2 605	206	2 470	210	2 458		
203	2 509	207	2 459	211	2 469		

UNIT NO. 13.

23-*B*	2 550
26-*B*	2 749

(NOTE: A small error in titration makes a relatively large error in the tabulated values herein.)

Thus the tests on Unit No. 11 in September—Nos. 105-109—average 25.92, and those in November—Nos. 111-122—average 27.98. In the former case the concentration of the nitrate was 1.5622; in the latter case it was about 1.449. Hence the titration in November, computed from that in September, should be

$$25.92 \times \frac{1.5622}{1.449} = 27.95$$

which agrees very well indeed with the average for November mentioned previously.

The concentration of nitrate in November was taken at about 1.45 grammes per liter so as to facilitate the determination of the approximate salt content of the normal head-water from the titration of a sample by simply dividing by 2. Concentrations computed in this

way, however, are subject to the error of titration. This error, for normal head-water, is in the neighborhood of 1% on the side of excess.

47.—Error 2.—Variability of Initial Content.—Whether the elimination of the error due to initial content is complete or only partial depends in large measure on whether this content is variable and on the method of taking samples.

In order to eliminate the error due to variability of content, it is necessary only to secure an average sample. Such an average must be a volume average and not a time average. However, as the velocity at any point in the cross-section at the head-water sampling station is nearly constant, a continuous sample taken at a uniform rate during any test, or part of a test, will represent a fair average for that point during the observation. If the variability is so great as to cause very different concentrations in different parts of the section at the same time, more than one sample will be required for the normal head-water. If, in addition, there is much variation in velocity from point to point of the cross-section, the concentrations of the samples from these points would have to be weighted in proportion to the relative velocities at the same points.

Fortunately, the latter precaution is seldom needed; a single head-water pump, operating continuously during a test, will suffice in the majority of cases. Indeed, the constancy of salt content can be checked by taking several continuous samples from each pump during a test, and noting that the special dilutions resulting from these samples all titrate substantially alike.

Table 7, Section 46, gives a list of normal head-water titrations observed during the tests.

48.—Error 3.—Variability of Standard Chemical.—If the concentration of the silver nitrate should vary during a series of titrations, it may introduce an error of any magnitude, depending on the extent and character of the variations. At first sight, it would appear that this error cannot be eliminated; but, on closer study, it will be apparent that if the methods advocated in this paper are followed, gradual changes may be eliminated by group titrations. Suppose, for example, that two chemists are each titrating the same test in two independent treatments. It will not matter if their silver nitrate solutions are different; they should arrive at the same result, notwithstanding, for titrations vary in magnitude in an inverse manner

with variations of concentration, being nearly in an inverse ratio thereto, and, as the methods advocated herein require that all titrations shall be approximately equal, they will be affected in any test by the same constant error, which, therefore, will cancel from the equations, because only the ratios of the titrations enter such equations.

If, therefore, the same number of duplicate samples of each solution are prepared, the samples may be titrated in groups containing one or more samples of each solution. Only one chemist should titrate any one group, and, during its titration, he should be sure to use the same instruments, the same silver nitrate solution, and the same indicator solution. He should even go so far as to approximate the same readings of his burette and other scales, in measuring his solutions. In fact, the burette should be filled nearly, if not exactly, to the zero of its scale before each titration, and should be proportioned to the work so that it will be almost emptied by each titration, with only a good practical margin to spare. This is on the side of accuracy and economy. The reason for this is more apparent when it is observed that, by following these directions, each titration of any group will be affected with the same constant error, including the personal error of the chemist, as only one chemist is permitted to titrate any one group of samples.

Not only should the samples be thus divided into groups, but each group must contain nearly the same number of samples of each kind, so that they may be titrated one kind after the other as rapidly as possible, with perhaps an extra sample of one kind, so that the titration of the group may begin and end with that same kind of sample.

For example, suppose a test has been run in which there are twelve tail-water samples, T_1, T_2, etc., four special dilutions, D_1, D_2, etc., and four salt solution samples, S_1, S_2, etc. Then, to eliminate the errors due to changes in the nitrate, the personal error of the chemist, and errors of the graduation marks on the burette, the preceding directions must be followed, and the groups may be arranged as follows:

GROUP TITRATIONS.

Group 1.....	T_1	D_1	T_2	S_1	T_3
Group 2.....	T_4	D_2	T_5	S_2	T_6
Group 3.....	T_7	D_3	T_8	S_3	T_9
Group 4.....	T_{10}	D_4	T_{11}	S_4	T_{12}

A Second Grouping.

$$\begin{array}{ccccc} T_1 & D_1 & T_2 & S_1 & T_3 \\ T_4 & S_2 & T_5 & D_2 & T_6 \\ T_7 & D_3 & T_8 & S_3 & T_9 \\ T_{10} & S_4 & T_{11} & D_4 & T_{12} \end{array}$$

There appears to be no urgent need for the second grouping, rather than the first, though some chemists may prefer it.

The second grouping might cause confusion in entering the record on the record sheets, with the possibility of making an error by entering a special dilution in a salt-solution space. This error, however, would be small if the directions are followed closely, as all titrations of the same group are nearly equal among themselves.

Each group should be titrated as rapidly as possible, by the same chemist, as previously explained, and in the order of reading from left to right, or the contrary. As a consequence of this procedure, it will be seen that gradual changes in the chemicals used, such as increasing concentration due to evaporation or changes in temperature, will not affect the results if the work is executed as outlined.

49.—Errors 4 and 5.—Indicator, and Excess of Standard.—The determination of the end point of any reaction is always affected by an error. Part of this error may be due to reactions between the indicator and standard solutions used and part may be due to the necessity of having an excess of standard in order to make distinct the indication of the end point. In either case, the titration, as determining the quantity of chlorine, or other chemical, is in error.

So far as determining the quantity of water, or other liquid, is concerned, however, it will not be in error if the method of special dilutions developed in the tests described herein is used. For such an excess of titration will affect all the titrations alike, as they are of nearly the same magnitude for substantially equal samples.

Indeed, it follows as a consequence of the methods laid down that constant errors of titration are such only as relate to the exact determination of the quantity of chemicals present. They are not errors at all as relates to the determination of volumes of liquids by the method of special dilutions.

50.—Error 6.—Uniformity of Mixture and Distribution of Salt in Tail-Water.—It seems that many engineers have an idea that failure to secure uniformity of mixture means failure to secure an

accurate test. That such is not the case may be readily seen from the following and similar considerations.

In the tests described there were two draft-tubes delivering the discharge from a pair of vertical twin runners. In some of the tests the distribution of salt between the upper and lower draft-tubes was nearly equal; in other cases there was as much as 15% and even 20% more in one draft-tube than in the other, as may be seen by examining the titrations of the tail-water samples which are collected in Part IV. Let it be required to determine the possible errors due to this cause:

It is clear that it is a volume average of concentration which is required for the value of c_2 in the equations. Therefore, if the concentration varies from point to point in the cross-section of the tail-race, the arithmetical mean of the concentrations of several samples taken at these points will not be an average concentration unless the velocity is uniform at all points, which is never the case. It is necessary, therefore, to determine the weighted mean, the weight of any concentration being the ratio of the quantity of water discharged at that concentration to the total quantity discharged during the test. Thus, if W_1 is the volume of water discharged by the upper draft-tube at the average concentration, c_1, and W_2 is that discharged by the lower draft-tube at the average concentration, c_2, then the weighted mean, or volume-average of the concentration for the test, will be

$$M = c_1 - \frac{W_2}{Q}(c_1 - c_2) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (138)$$

where Q is the total discharge of both draft-tubes during the test.

Take, now, the most unfavorable case, where the concentrations in the upper and lower draft-tubes are, respectively, 55 and 45 c.c. of silver nitrate. Further, suppose that the upper draft-tube discharges 20% more water than the lower one. This, of course, is a far greater difference than can reasonably be imagined, as the runners are exactly alike. However, with this excessively unfavorable assumption, observing that the discrepancy between the discharges is about 10% of the total discharge, we have,

$$M = 55 - \frac{50}{110}(55 - 45) = 50.45$$

as against

$$\frac{55 + 45}{2} = 50$$

for the arithmetical mean. Thus, the error introduced is less than 1% under assumptions which must be more unfavorable by far than the actual conditions under which the test was executed. It would be much more like the actual conditions to assume that the upper runner differed from the lower by only about 5%, in which case the weighted mean would be

$$M = 55 - \frac{50}{102.5} \times 10 = 50.12$$

which does not differ from the arithmetical mean by as much as ¼ of 1 per cent. This is probably too large an error, as the gates of the two water-wheels are undoubtedly set so as to cause a nearer approach to equality of discharge than that last supposed.

It follows, from the preceding comparisons of the arithmetical and weighted means, that, if we have anything approaching the true weights of the concentrations observed in the tail-race, we can compute an average concentration which will differ from the true average by an error which is measured by only a few hundredths of 1%, and this, too, where there are considerable variations among the observed concentrations and velocities.

The foregoing remarks apply particularly to the case of two draft-tubes. In the case of a single draft-tube, or tail-race, there are likely to be greater variations of relative velocity from point to point than has been supposed to exist between the two draft-tubes. Thus, this case requires a study of the errors similar to that for the case of a pair of draft-tubes, but it will usually be found that the distribution of salt over the section of a single draft-tube is so much better than it is between two draft-tubes that the resulting error is even less than those just discussed.

Take, for example, one of the best tests on Unit No. 13, such as Test 41. The detailed distribution of concentration in the draft-tubes is shown by Table 8.

TABLE 8.—*Distribution of Concentration in Test 41, as Shown by the Titration of the Eighteen Tail-Race Samples.

	West Half.			East Half.		
Upper draft-tube	53.20	54.45	53.92	53.85	54.40	54.94
	53.45		54.70		54.85	
Lower draft-tube	54.05	53.85	53.80	54.05	54.30	53.25
		53.90		54.20		54.55

*The units are cubic centimeters of silver nitrate consumed in titration.

It will be seen at once that the lowest concentration is 53.20, and the highest is 54.94, or a maximum variation among samples of only a trifle more than 3% of the average. It follows that the arithmetical mean, 54.095, is certainly within 1½% of the truth, and the probable error is only a small fraction of 1%, the assumption being that the average distribution of chlorine among the various samples is uniform, and that the figures of Table 8 vary by accidental errors due to one cause or another.

If, on the other hand, it is considered that the variations in Table 8 are real variations of concentration, then we must seek to weight each concentration by the relative velocity at the point where it is observed. It will be better engineering, however, and far more economical of time, where a number of tests are to be thus discussed, to divide the upper and lower draft-tubes into east and west halves, thereby dividing the tail-race into quarters, and then to weight the mean concentration of each quarter with the mean relative velocity of that quarter. It may be remarked, also, that we are now regarding the tail-race as the single discharge passage from a turbine unit. This is reasonable, as the concentrations are so nearly uniform that there is no material variation of the mean concentrations between the upper and lower halves of the tail-race. Of course, attention might be confined to either upper or lower draft-tube, if one wished to be very consistent. The result will be the same in either case.

The mean concentrations of the four quarters may then be arranged as follows:

COMPARISON BY QUARTERS.

53.944	54.510
53.900	54.070

By thus taking means for the quarters, the maximum variation among the four averages for the samples of each quarter has been diminished to 0.61 c.c., or only a trifle more than 1%, as compared with 3% when the samples are taken separately. It may well be concluded, then, that the arithmetical mean of all samples is well within ½ of 1% of the truth, at least in this particular case, and, consequently, that the weighted mean will be within a small fraction of 1% of the truth.

It will also be of interest to compare the upper and lower and east and west halves of the tail-race concentrations, thus:

COMPARISON BY HALVES.

Mean of nine samples, upper half	54.196
lower half	53.994
Mean of nine samples, west half	53.922
east half	54.290

These results show that the variation of average samples has again been reduced, and we note that the variation for the upper and lower halves is only 0.2 c.c., and it is less than 0.4 c.c. for the east and west halves. Owing to this narrowing of the variation, it is apparent that the probability is very high, in this particular case, that the arithmetical mean is not in error by more than $\frac{1}{4}$ of 1% at most, and a mean derived from anything like the true weights cannot be in error by as much as $\frac{1}{10}$ of 1 per cent. All this may be proved, with large margins in favor of the less formal conclusions, by a simple application of the principles of probability, but would not add anything to the probability of the fact which we show, namely: that the variability of concentration is not such a serious matter as many have supposed, and, above all, that it is quite possible, in testing large-capacity turbines operating on low heads, to secure remarkably uniform distributions, even where there are two water-wheels operating on one shaft and discharging through two separate draft-tubes.

By referring to Fig. 37 the proper weights for the concentrations of the four quarters may be shown to be about as follows:

	West.	East.	Mean.
Upper	0.30	0.20	0.50
Lower	0.33	0.17	0.50

Therefore, the weighted mean for this test may be computed very closely as follows:

$$0.30 \times 53.944 = 16.183$$
$$0.33 \times 53.900 = 17.787$$
$$0.20 \times 54.510 = 10.902$$
$$0.17 \times 54.070 = 9.192$$

Weighted mean.. 54.064

A comparison of the weighted and arithmetical means may be made, thus:

Arithmetical mean	= 54.095
Weighted mean	= 54.064

which are seen to differ by less than $\frac{6}{100}$ of 1%, with the probability that the weighted mean is in error by even less than this, as it has been repeatedly shown in the chemical laboratory that the means from sets of six or eight equal samples will titrate within $\frac{5}{100}$ of 1% of each other.

This latter, of course, is an accidental error as distinguished from the systematic error due to erroneous weighting which has just been discussed. This distinction must be carefully made in criticising accurately made tests; and the residual error, resulting from these two classes, must be shown not to be greater than a certain quantity before any conclusions can be drawn as to the accuracy of the tests.

By analyzing the salt distribution in the tail-race in a manner similar to the foregoing, a close estimate of the degree of accuracy attained in the weighted mean for the tail-race concentration in any particular test can be made, and thus the effect on the final result of the test may be ascertained. It is readily seen, therefore, that, so far as mixture and distribution of salt are concerned, the best tests are well within the desired degree of accuracy of $\frac{1}{10}$ of 1 per cent.

Of course, it is assumed, in making this last statement, that the weights are not vastly incorrect, although there is a very wide margin of error permissible. For example, by taking the weights twice as great, relatively, on the west side of the race, as they are in the foregoing table of weights, we would have:

	West.	East.	Half Tube.
Upper.....	0.375	0.125	0.50
Lower.....	0.400	0.100	0.50

which leads to the following calculation of the weighted mean, on the assumption that the weights used in the computation of the mean of 54.064, above, are only half as great relatively on the west side as they should be:

$$0.375 \times 53.944 = 20.229$$
$$0.400 \times 53.900 = 21.560$$
$$0.125 \times 54.510 = 6.814$$
$$0.100 \times 54.070 = 5.407$$

Weighted mean, based on assumed weights..... 54.010

Thus, if the weights used in obtaining the mean, 54.064, are only half as large on the west as they should be, there would be introduced an error in the weighted mean of only 0.1 of 1 per cent.

Now, Fig. 37 is based on current-meter observations in the tail-races of the units tested, and, though the current-meter work is to be regarded as only a rough field operation, without correcting the meters for errors due to turbulence, it is difficult to suppose that the errors in the relative velocities, east and west, can be more than 15 or 20 per cent. Hence, it is concluded that no serious error results in the best tests by reason of errors in weighting the concentrations of salt in the tail-race samples.

Finally, it may be stated that the ultimate determination of the efficiencies of the units justifies the last conclusions, not only when tests of the same unit are compared with one another, but also when the tests of Units Nos. 12 and 13 are compared. These two units being exactly alike, their efficiencies should be equal, with only such differences as may be due to slight differences in hydraulic conditions, adjustment, packing, lubrication, etc.

51.—Distributing Pipes and Centrifugal Pump.—As explained in the description of the testing plant, Part III, the salt solution was injected into the suction of an 8-in. centrifugal pump, which discharged into a header feeding twelve 2-in. pipes. Six of these 2-in. pipes passed down the stop-log slot on the west and six on the east side of the head-race. These pipes are numbered from 1 to 12, the odd numbers being west, the even numbers east, and progressing downward.

There were six horizontal 2-in. pipes extending across the head-race, each perforated with three series of holes, one series along the top element discharging vertically upward, one series along the bottom element discharging downward, and the third series along the up-stream element discharging against the current. A pair of differential pulleys—one such pulley being at each side of the head-race—facilitated the adjustment of these six horizontal distributing, or sprinkling, pipes, so that they divided the rectangular cross-section of the race into six equal horizontal strips, with a pipe in the horizontal median of each strip.

The twelve distributing pipes from the header were connected by 2-in. hose to the ends of the risers leading to the horizontal sprinkling pipes in pairs, so that Nos. 1 and 2 supplied the upper, or No. 1, sprinkler, with salt solution; Nos. 3 and 4 supplied the next lower, or No. 2, sprinkler, and so on downward.

In the tests on Unit No. 13 these horizontal sprinklers were open throughout their length, but in some of the later tests a block of wood, or plug, was introduced at the middle of the pipe in the center line of the head-race.

The relative discharge from each of the various parts of the distributing system was controlled by gate-valves and gauges connected to the distributing pipes, 4 or 5 ft. below the point where they branched from the header. At these points there was also provided a pet-cock on each pipe, so that samples of the mixtures being pumped into the different pipes could be secured.

During the course of a number of the regular tests, samples were taken from these cocks and titrated so as to obtain an estimate of the relative quantities of salt passing through each of the twelve pipes leading from the header.

In this way it was proved conclusively that the method of introducing the salt into the centrifugal pump effected a perfect mixture with the water being raised from the canal by the pump, so that the discharge was a salt solution of perfectly uniform strength in all the pipes at any given time. The experiments also enable one to calculate the discharge of the centrifugal pump, although no effort was made to observe the power consumed by the pump or to compute its efficiency.

The observed titrations, uncorrected for temperature effects and calibrations of glassware, for two such tests as have been described, are given in Tables 9 and 10, with approximate calculations for the

TABLE 9.—SHOWING UNIFORMITY OF MIXTURE DISCHARGED FROM THE 8-INCH CENTRIFUGAL PUMP IN TEST *V*, UNIT NO. 13, AUGUST 10TH, 1914.

The figures are in cubic centimeters of silver nitrate consumed in titration up to the end point.

Horizontal pipe No.	West.	East.
1	25.75	26.15
2	25.95	25.85
3	25.85	25.80
4	25.85	25.95
5	25.75	25.75
6	25.80	25.70
Means	25.825	25.867
Average of all		25.846
Average, less 0.07 c.c., by Table 5, Section 43		25.78

discharge of the pump in all the tests where distributing-pipe samples were taken. The practical equality of the titrations of the twelve samples taken from the distributing pipes is very evident.

The value of v_2 for these twelve titrations, therefore, is about 25.78. A sample consisted of 10 c.c. pipetted from a stock solution made up by taking 20 c.c. of the discharge from a given pipe and diluting to 500 c.c. with distilled water. Thus, by multiplying the observed titration by $100 \times 25 = 2\,500$, there results a number which is, theoretically, what the titration of 1 liter of the actual discharge would have been. The uncorrected concentration of the salt solution is 544 900 and the concentration of the normal head-water, as usual, is about 25.72. Therefore, the calculation of the discharge of the pump for Test *V*, with considerable approximation, is

$$v = 544\,900$$
$$v_1 = 25.72$$
$$v_2 = 2\,500 \times 25.78 = 64\,450$$
$$q = 0.24 \text{ (from the record)},$$

and

$$Q = 0.24\ \frac{544\,900}{64\,450} = 8.46 \times 0.243 = 2.06 \text{ cu. ft. per sec.}$$
$$= 2.06 \times 60 \times 7\tfrac{1}{2} = 928 \text{ gal. per min.}$$

The elevation of the surface of the canal water was 198.6, and that of the center of the pump was about 217, so that the suction lift was 18.4 ft. The speed of the pump was about 600 rev. per min.

TABLE 10.—MIXTURE DISCHARGED FROM 8-INCH CENTRIFUGAL PUMP IN TEST 26, UNIT NO. 13, AUGUST 26TH, 1914.

Horizontal pipe No.	West.	East.
1	28.09	28.12
2	28.15	28.07
3	28.13	28.15
4	28.10	28.07
5	28.10	28.10
6	28.20	28.25
Means	28.128	28.127

Average of all.....28.13
Average, less 0.06 c.c. by Table 5, Section 43.....28.07

Computation with approximate data from test:

$v = 528\,600$

$v_1 = 25.72$

$v_2 = 2\,500 \times 28.07 = 70\,180$

$q = 0.24$ (from record),

$$Q = 0.24 \frac{528\,600}{70\,180} = 0.242 \times 7.54 = 1.82 \text{ cu. ft. per sec.}$$

$$= 1.82 \times 60 \times 7.5 = 820 \text{ gal. per min.}$$

Elevation of surface of canal water = 195.8

Suction lift = 217 — 195.8 = 21.2 ft.

Thus the effect of the additional suction lift is apparent in the decreased discharge, the pump running at about the same speed as before.

Table 11 gives a summary of all the titration tests of the discharge of the centrifugal pump, with foot-notes explaining certain apparently anomalous results due to varying conditions of operation.

TABLE 11.—CENTRIFUGAL PUMP DISCHARGE DURING VARIOUS TESTS.*

Data, uncorrected for temperature.†	Symbol.	TEST NUMBER.						
		D	*E*	*S*	*V*	26	43	118‡
Titration of salt solution, in cubic centimeters	v	502 700	581 000	545 700	544 900	528 600	542 500	621 400
Titration of head-water, in cubic centimeters	v_1	25.28	25.21	25.56	25.72	25.72	25.94	28.05
Titration of pump discharge, in cubic centimeters	v_2	62 000	81 650	78 150	64 450	70 180	74 680	73 480
Ratio of dilution	R	8.12	7.12	6.98	8.46	7.54	7.27	8.48
Rate of discharge of salt solution, in cubic feet per second	q	0.257	0.281	0.283	0.243	0.242	0.241	0.242
Suction lift, in feet	.....	16.9	17.5	17.7	18.4	21.2	20.7	19.3
Discharge of pump, in gallons per minute	Q	936	901	735§	928	820	788	924

NOTE.—The effect of a change in suction lift is very apparent in these tests. The readings of gauges on distributing pipes can be found in Tables 46, 47, 48, and 49. No exact pressures are available. See Section 101.

* The discharge of this pump should be added to the turbine discharge determined by the current meters in tests where the pump was running, as the pump discharge was shunted around the meter section.

† Although the data are uncorrected for effects of temperature, they are sufficiently accurate for the purpose of this table.

‡ Test 118 was on Unit No. 11. All the other tests of this table were on Unit No. 13. Unit No. 13 was equipped with a direct-current generator, but No. 11 was equipped for alternating current.

§ In Test *S*, all six of the feeders to sprinkling pipes on west side of head-race shut down, which accounts for the low discharge of pump in this test.

‖ In Test 118 another motor was used, which ran the pump at speeds considerably greater than 725 rev. per min. Two of the sprinklers, however, were shut down.

The principal information to be gained from these tests is that a centrifugal pump appears to be an excellent mixer, and, consequently, is especially susceptible to the chemical method of turbine testing. The pump was an old one, and the runner was in poor condition, so that the efficiency was undoubtedly low. Neither was the speed anywhere near high enough in most of the tests, all of which facts, undoubtedly, added to the mixing qualities of the pump.

52.—Distribution Tests.—It was shown in Section 51 that the distribution of salt among the twelve distributing pipes was uniform. The results of several tests on Unit No. 13 to determine the distribution of salt in the tail-race arising from the separate discharge of each sprinkling pipe and also the results of tests to ascertain the time required after starting a test to establish steady conditions of distribution in the cross-section of the tail-race will be discussed in this section.

The former of these matters was the subject of investigation in Test *M*, July 25th, 1914. The salt-solution tank was filled with a strong solution, the usual samples of which titrated as follows:

SALT SOLUTION, TEST *M*.

54.80	54.85
54.75	54.75
54.80	54.80
54.80	54.80
219.15	219.20
Mean..... 54.79	

After this each of the six sprinkling pipes was opened in succession for about 5 min., but with scarcely sufficient time intervening to permit the establishment of steady conditions in the pipes and tail-race. The sample bottles at the tail-race sampling pumps were changed each time so that separate sets of samples were obtained for each pipe. There were thus obtained 84 tail-race samples, 14 representing the relative distribution of salt in the tail-race for each of the six sprinklers.

The numbers of the pipes progress downward, so that Pipe 1 was the uppermost and Pipe 6 the lowest in the head-race distributing system. The tail-race sample pipes were in the cross-section at the stop-log slot of Unit No. 13, just outside the power-house. Some time

afterward this section was abandoned on account of the difficulty of keeping the ¼-in. pipes in repair.

TABLE 12.—TESTS *M*.—DISTRIBUTION OF SALT IN TAIL-RACE. TITRATIONS OF THE INDIVIDUAL TAIL-RACE SAMPLES.

(The unit is 0.1 c.c. of silver nitrate, and each number represents the titration of ½ liter.)

NUMBER OF SPRINKLING PIPE OPEN, TESTS *M*.											
1.		2.		3.		4.		5.		6.	
W.	E.	W.	E.	W.	E.	W.	E.	W.	E.	W.	E.
322	295	306	285	230	292	170	210	138	144	136	138
378	270	298	288	272	270	169	200	136	144	137	142
337	269	287	269	212	264	166	206	120	152	138	156
152	167	160	180	201	231	259	258	300	255	282	283
187	137	147	153	192	212	263	267	318	300	322	321
136	185	146	150	202	204	268	272	317	208	313	329
267		315		266		183		188		144	
131		145		200		251		...		347	

NOTE.—The figures below the twelve regular samples represent, respectively, the relative concentrations at the center lines of the upper and lower draft-tubes, and were determined from samples taken from Pumps 41 and 42.

Table 12 shows the actual titrations of samples in the tests. These should be reduced by 13.5 c.c., that is, by 135 tenths, the titration of ½ liter of normal head-water, in order to secure numbers representing the relative salt content of the tail-water at the various sampling pipes resulting from the introduction of salt solution through the particular sprinkling pipe which is open in any particular test. It will make the different tests more directly comparable with each other, however, to reduce these resulting relative concentrations to percentages of the mean concentration for the whole tail-race by dividing each by this mean. The result of this compilation is given in Table 13, which, therefore, represents quite accurately the relative distribution of concentration in the tail-race due to each of the six sprinkling pipes.

From Table 13 it is apparent that Pipes 1 and 2 deliver nearly all their salt supply to the upper draft-tube, and Pipes 5 and 6 supply principally the lower draft-tube. Pipes 3 and 4 divide their supplies much more evenly, but Pipe 3, being above the middle of the cross-section in the head-race, naturally supplies a greater proportion to the upper draft-tube, and Pipe 4, being below the middle supplies a

greater proportion to the lower draft-tube. All this might easily have been anticipated, but Table 13 is very valuable, as it enables one to secure a uniform distribution in the tail-race by adjusting the valve on the proper pipe in the head-race. It was by using this table that good distributions were secured in the tests.

TABLE 13.—TESTS *M*.—SEPARATE DISTRIBUTIONS OF SALT IN TAIL-RACE OF UNIT NO. 13 DUE TO EACH OF THE SIX SPRINKLING PIPES IN THE HEAD-RACE BASED ON TWELVE SAMPLES OF TAIL-WATER.

(The figures represent percentages of the mean values of the concentrations at the twelve sampling pipes.)

NUMBER OF SPRINKLING PIPE OPEN, TESTS *M*.											
1.		2.		3.		4.		5.		6.	
W.	E.	W.	E.	W.	E.	W.	E.	W.	E.	W.	E.
201	172	196	172	98	162	39	82	3	10	2	3
262	146	182	175	142	189	38	71	2	11	2	8
217	144	175	154	80	133	34	79	−18	20	3	23
18	34	29	52	68	100	137	135	196	142	164	165
2	2	14	21	59	80	141	146	217	196	209	207
½	0	13	17	69	71	146	151	216	205	199	216

Table 12 gives additional evidence in favor of the conclusions of Section 50, that variations of distribution are not vitally important where the water in the tail-race is all flowing outward, that is, out of the tail-race. This is made clear by Table 14, a compilation of the average values of that part of the tail-race concentration in Tests *M* which is due to the separate sprinkling pipes. Thus, there were six tests compared in Table 12; taking the arithmetical average in each

TABLE 14.

Number of sprinkling pipe open.	Salt rate, in liters per second.	Average equivalent titration, in tenths of cubic centimeters.	Ratio of titration to salt rate.
1	1.53	93	61
2	1.63	87	53
3	1.62	97	60
4	1.61	90½	57
5	1.71	84½	49
6	1.78	90	50
Means	1.65	90.3	55

case and deducting 13.5 c.c., that is, 135 tenths, Table 14—a table of averages—has been compiled.

The salt rate is simply the rate of discharge of the salt solution during the times the samples were being taken in the various tests (about 5 min. in each test). The strength of the salt solution is stated in tabular form at the beginning of the section. A close agreement between the salt rate and the mean concentration in the tail-race need not be expected, nor required, in these tests, as the period of the tests was too short for such a degree of accuracy, and it is doubtful whether sufficient time was allowed between tests. Moreover, it does not appear that the salt rate was maintained anywhere near constant during any test, so that it is easy enough to account for these variations among the mean concentrations and the ratios between them and their corresponding salt rates. These facts do not detract in the least from the importance of the conclusion which may be drawn from the fact that the average equivalent titration per liter of salt rate varies by only 10 or 12% from the mean, 55, as a consequence of opening successively the six sprinkling pipes in the head-race, thereby varying the relative intensities of concentration in the various parts of the tail-race due to the introduction of salt solution from nothing to more than 2½ times the mean concentration during the test, as shown in Table 13. The conclusion is the same as that of Section 50, namely, that large variations in the concentration of salt in the tail-race make relatively small variations in the average concentration computed therefrom, even though there are considerable variations of velocity in the tail-race, as in the regular turbine tests where the velocity variations were from 1 or 2 ft. per sec. on one side to 4 or 5 ft. on the other.

The ratios above computed are somewhat improved, especially for all tests after the first, which latter took place just after opening the salt valve, by weighting the concentrations according to the scheme of weights given in the diagram, Fig. 37.

Thus, the weights from that diagram for the four halves of the two draft-tubes for Test *M*, in which the gate was set at 14.5, are

Upper west	0.302	Upper tube....0.50
Upper east	0.198	
Lower west	0.320	Lower tube....0.50
Lower east	0.180	

Possibly the discharge of the upper draft-tube should have slightly more weight than the lower one, as it is supposed that the gate of the upper turbine was slightly more open than that of the lower, but the diagram is based on equal weights.

Weighting the averages for each of the four quarters of the tail-race, accordingly, Table 15 has been compiled.

TABLE 15.—TESTS *M*.—WEIGHTED MEAN TITRATIONS OF HALF-LITER TAIL-RACE SAMPLES, PER LITER PER SECOND, OF SALT SOLUTION DISCHARGED FROM EACH OF THE SIX SPRINKLER PIPES IN THE HEAD-RACE, AND RATIO OF THE WEIGHTED MEAN TO THE ARITHMETICAL MEAN.

(Compiled from Tables 12 and 13.)

(The units are cubic centimeters of silver nitrate per liter per second of salt solution for half-liter tail-water samples.)

Number of sprinkler open.	Salt-rate, in liters per second.	Weighted mean, per liter per second, of salt solution.	Arithmetical mean, per liter per second, of salt solution.	Ratio of weighted to arithmetical means.
1	1.53	6.28	6.08	1.03
2	1.63	5.34	5.34	1.00
3	1.62	5.80	5.99	0.97
4	1.61	5.47	5.62	0.97
5	1.71	4.97	4.93	1.01
6	1.78	5.05	5.05	1.00
Average of all six		5.48	5.51	
Average of last five		5.33	5.39	

The last column might be computed by weighting the ratios given in Table 13, instead of taking the quotients of the weighted and arithmetical means directly as in Table 15.

It is seen from Table 15 that 1.03, the ratio for the first of Tests *M*, is not in the direction of improving the weighted mean for this test, and that the last ratio should be greater than, instead of equal to, unity. Otherwise, all the other ratios of the last column tend to reduce the variations among the weighted means, decreasing the arithmetical means which are larger than the average and increasing those which are smaller. For example, the maximum variation of the weighted mean is $5.80 - 4.97 = 0.83$, and that of the arithmetical mean is $5.99 - 4.93 = 1.06$. Had the tests been of longer duration,

this result would have been materially improved, and the variations among the values of the weighted mean would have tended to vanish altogether with increasing care in the experiments. Of course, we depend here, also, on the current-meter tests, on which Fig. 37 is based. There are no doubt errors of some magnitude in the current-meter work, but the error introduced into the weights has been shown not to be serious for the purpose of computing the discharge.

TABLE 16.—OBSERVED AND WEIGHTED CONCENTRATIONS OF SALT IN TAIL-RACE DUE TO THE SEPARATE DISCHARGE FROM EACH QUARTER OF THE HEAD-RACE SPRINKLING SYSTEM, TESTS 101-104, UNIT No. 11.

(The unit is the titration of 100 c.c. unevaporated tail-water. $AgNO_3 = 1.56$ grammes per liter.)

Test No.	Half of tail-race.	Actual titration of 100-c.c. sample of tail-water due to salt from tank only.		Weighted titration.	
		W.	E.	W.	E.
101 Upper west quarter of sprinkling system open	Upper...	8.99	12.35	3.49	1.39
	Lower...	8.68	11.19	3.27	1.38
	Means...	10.30		9.53	
102 Lower west quarter open	Upper...	3.28	4.52	1.27	0.51
	Lower...	13.27	15.63	5.00	1.92
	Means...	9.18		8.70	
103 Lower east quarter open	Upper...	2.71	3.19	1.05	0.86
	Lower...	13.32	12.25	5.02	1.51
	Means...	7.87		7.94	
104 Upper east quarter open	Upper...	11.79	10.15	4.57	1.15
	Lower...	2.53	2.68	0.95	0.33
	Means...	6.79		7.00	

Several tests similar to those just discussed were made on Unit No. 11. These are Tests 101-104. In each case the sprinkler pipes of only one-quarter of the head-race were opened, the pipes in the other three-quarters remaining closed. The effect on the distribution is shown in Table 16.

In this case only 100 c.c. of tail-water constituted a sample, and this was titrated directly without evaporation. Thus, the titrations of these tests and those of Tests *M* are not directly comparable, the titrations of 100 c.c., unevaporated, being relatively considerably larger than those for 500 c.c. evaporated. Moreover, the titration of 100 c.c. of normal head-water directly requires from 3.50 to 4.50 c.c., which is relatively much larger than for 100 c.c. of tail-water, the comparable value probably being 3.50, which is adopted for the following discussion. This value has also been taken high on account of the fact that a 100-c.c. sample of tail-water will titrate several per cent. higher, relatively, than the half-liter samples of Tests *M*. By thus having the normal head-water titration too high, the remainder, after subtraction, will be decreased several per cent., which will tend to counteract the over-titration.

The weights for the four quarters of the tail-race for Tests 101-104 on Unit No. 11, from Fig. 39, are:

	West.	East.
Upper	0.387	0.113
Lower	0.377	0.123

When these weights are applied to the actual titrations of Table 16, the corresponding weighted titrations result.

By dividing these titrations by the salt rate and multiplying by 5 to reduce the titration to ½ liter, we should be able to make some comparison between the distribution tests on Unit No. 13, summarized in Table 15, and those on Unit No. 11. Thus Table 17 has been compiled.

These rates of titration are considerably larger than those of Tests *M*. This is due to the fact that the discharge was considerably larger in Tests *M* than in Tests 101-104. The titrations should also be proportional to the strength of the salt solution. The salt solution was not titrated, but 100 c.c. of the tail-water was titrated to about 11.8 c.c., unreduced by the initial salt content, which corresponds to

59 c.c. per ½ liter. The hydrometer readings for the salt solution in Test 105 was 1 180, and for Tests 101-104 it was 1 178, so that the salt solution was of the same strength as in Test 105 when the titration was at an average of 53.5. The titration of the salt solution in Tests *M* was 54.8, so that, admitting errors of 2 or 3%, the two tests will be comparable when the difference of discharge is taken into account. The discharges in the two cases are about as follows:

Tests *M* 1 730 cu. ft. per sec. = 49 000 liters per sec.
Tests 101—104 1 510 " " " " = 42 700 " " "

TABLE 17.—TESTS 101-104.—WEIGHTED MEAN TITRATIONS PER LITER PER SECOND OF SALT SOLUTION DISCHARGED FROM EACH QUARTER OF THE SPRINKLER SYSTEM IN HEAD-RACE OF UNIT NO. 11 FOR 100-C.C. SAMPLES OF TAIL-WATER, AND RATIO OF THE WEIGHTED MEAN TO THE ARITHMETICAL MEAN.

Quarter of head-race treated with salt solution.	Discharge of salt solution, in liters per second.	Weighted mean titration. Table 16.	Arithmetical mean. Table 16.	Ratio of weighted to arithmetical mean.
Upper west (101).....	7.00	6.80	7.36	0.92
Lower west (102).....	6.85	6.35	6.70	0.95
Lower east (103).....	6.80	5.84	5.79	1.01
Upper east (104).....	6.80	5.14	5.00	1.03
Mean for first three tests.....		6.33	6.62	
Means for all tests.....		6.03	6.21	

The reader's attention has been called to the fact that the first and last of Tests *M* were questionable, and it is now also stated that the salt solution was exhausted in the middle of Test 104, so that this test was cut short, with the result that the incident casts doubt on the weighted mean for this test in Table 17. Cutting out these questionable tests, the average variations of the weighted and arithmetical means in Table 17 are, respectively, 96 ÷ 6.33 = 15% and 1.57 ÷ 6.62 = 24 per cent. Thus the variation of weighted values from their mean is only 7½%, although the variation of average titrations in the four quarters of the tail-race is, in some cases, 4, or 5, to 1.

To make a comparison of the mean titrations per liter per second of discharge from the salt-solution tank in Tests *M* with those of Tests 101–104, therefore, we may with some degree of approximation take the product of the titrations by the discharges of the turbines

in the two cases. If these products are doubled they will represent the theoretical titration of the entire discharge of the turbine for 1 sec. when the salt rate is 1 liter of solution per sec., the density of this solution being about 1.18. The results are:

Middle 4 of Tests *M* $2 \times 49\,000 \times 5.39 = 528\,000$ c.c.;
First 3 of Tests 101–104 $2 \times 42\,700 \times 6.33 = 540\,000$ c.c.;

which agree reasonably well for the character of work in the distribution tests. The agreement would have been even better had the middle pair, which are the more trustworthy, of Tests 101–104, been taken instead of the first three. Moreover, these figures are just about what 1 liter of salt solution would give when titrated in the usual manner. For example, the titrations of 10 c.c. of a 100 to 1 dilution of salt solution with distilled water in the two tests have been stated to be

Tests $M = 54.8$, or, 548 000 per liter;
Tests 101–104 $= 53.5$, or, 535 000 per liter.

Therefore, the relative variations of the weighted values from their means in Tests *M* and 101–104 are, respectively,

for Tests *M* $\frac{1}{2}$ of $0.83 \div 5.39 = 8\%$;
for Tests 101–104 $\frac{1}{2}$ of $0.96 \div 6.33 = 7\frac{1}{2}\%$;

which may be attributed in some measure to the variations of the concentrations at different parts of the tail-race, as shown by Tables 13 and 16, in which they are, respectively,

for Test *M*, from 2 to 1 to 100 to 1, in Unit No. 13;
for Tests 101–104, from $1\frac{1}{2}$ to 1 to 5 to 1, in Unit No. 11.

53.—Precision Inferred from Distribution Tests.—An estimate of the order of precision of the regular tests, as to constant errors due to imperfect mixing, may be made from the foregoing discussion. That is to say, we may infer approximately the degree of error in the average concentrations, determined on very carefully conducted tests, from the errors observed in the concentrations of very hasty rough tests.

Without taking extreme values, the following estimate of the variations of concentration of salt in the tail-water in the different quar-

ters of the tail-race, as shown by Tables 13 and 16 for Tests M and 101-104, respectively, are:

Tests M:		Test:	
1.	35 to 1	101.	2 to 1
2.	10 to 1	102.	5 to 1
3.	2 to 1	103.	5 to 1
4.	4 to 1	104.	5 to 1
5.	15 to 1		
6.	84 to 1		
Mean	25 to 1	Mean	4 to 1

Now, if it is assumed that the precision varies directly with the foregoing mean variation, it will follow that a variation in concentration of n to 1 should, as an average, result in a precision giving rise to a systematic error of

$$\frac{8\,(n-1)}{24}\% \text{ for Tests } M, \text{ Unit No. 13} \quad (139)$$

and an error of

$$\frac{7\frac{1}{2}\,(n-1)}{3}\% \text{ for Tests 101–104, Unit No. 11} \quad (140)$$

We have no distribution tests for Unit No. 12, but it may be said that conditions on this unit were intermediate to those on Units Nos. 11 and 13, even to the fact that Unit No. 12 is situated between the other two. Variations of concentration in certain tests on Units Nos. 11 and 13 are shown in Table 18.

Thus, by Equations (139) and (140), we have as the error due to erroneous weighting of concentrations in the best tests on Unit No. 13, by this method,

$$\frac{8 \times 0.045}{24} = 0.015\%,$$

and, for the tests on Unit No. 11, the error is

$$\frac{7\frac{1}{2} \times 0.165}{3} = 0.41 \text{ per cent.}$$

These results are somewhat better than merely relative. They show that the error due to imperfect mixture is very small in the case of Unit No. 13, being negligible, in fact, and that we may expect errors on Unit No. 11 of an appreciable fraction of 1 per cent. The error in the final location of a plotted curve, however, should be less still than these approximate values, which are also to be considered too high, as they undoubtedly contain accidental components.

TABLE 18.—APPROXIMATE MAXIMUM VARIATIONS OF CONCENTRATION OF SALT IN THE FOUR QUARTERS OF THE TAIL-RACE IN CERTAIN TESTS ON UNITS NOS. 11, 12, AND 13.

UNIT NO. 11.		UNIT NO. 13.		UNIT NO. 12.	
Test No.	Variation.	Test No.	Variation.	Test No.	Maximum variation.
105	1.34	22	1.02	200	1.23
106	1.06	23	1.06	201	1.06
107	1.20	26	1.08	202	1.13
108	1.06	32	1.02	203	1.27
109	1.06	33	1.04	204	1.26
111	1.18	39	1.14	205	1.48
112	1.20	40	1.02	206	1.19
113	1.26	41	1.01	207	1.15
114	1.24	42	1.01	208	1.25
115	1.20	43	1.07	209	1.23
116	1.15	44	1.03	210	1.06
117	1.15	45	1.03	211	1.05
118	1.06	46	1.05	212	1.07
119	1.12				
120	1.05				
121	1.23				
122	1.19				
Means.....	1.165		1.045		1.187

54.—Time Required to Establish Steady Flow of Salt.—An idea of the variation of concentration for 2-min. averages may be gained from the results of taking a sample every 2 min. from one of the tail-race sampling pumps in Test *A*, July 11th, 1915. The first sample was taken just in time to avoid capturing any salt, immediately before opening the salt-solution valve in the head-race. The samples titrated as follows, in order of number:

1	1305	5	5160	9	5060	13	4930
2	5305	6	5005	10	4920	14	5430
3	5065	7	4995	11	4700	15	5105
4	5090	8	4990	12	4700	16	5100

Units are in hundredths of cubic centimeters titration per ½-liter sample evaporated.

The variations of the concentration from interval to interval are due partly to variations of the rate of introducing the salt solution and partly to those irregularities of the discharge of the turbine which may be expected for intervals as short as 2 min. That is, there are surges in the head-race which produce irregularities of concentration, even

when the salt rate is constant. There was not a sufficiently close connection between time and concentration in Test *A* to admit of a more complete discussion. The important fact to be noticed in this test is that it required but one time interval, 2 min., for the concentration to jump from that of normal head-water, 13.05 c.c., to fully concentrated tail-water, 53.05 c.c. The reason that this latter titration is considerably larger, with only one exception, than all the others is that the salt-solution valve is first fully opened and then adjusted to some predetermined salt rate.

In Test *D* there is a better connection between the time of starting the salt and the time of steady concentration. With possible errors of ½ min., the results are as given in Table 19.

TABLE 19.—TWO-MINUTE, HALF-LITER, TAIL-WATER SAMPLES IN TEST *D*.

Time, P. M. Hour and minute. H. M.	Salt rate during interval preceding, in liters per second.	Titration of tail-water sample, in hundredths of cubic centimeters.	Titration, less 1270, for normal head-water.	Remarks.
2-54	....	.....	.....	Began taking 2-min. samples.
56	0.00	1 270	0	
58	0.00	1 275	5	Started salt solution.
3-00	6.41	1 350	80	
2	6.68	4 200	2 930	
4	6.90	4 815	3 545	
6	7.08	4 620	3 350	
8	7.28	4 725	3 455	
10	7.48	5 080	3 810	
12	7.68	5 150	3 880	
14	7.79	5 195	3 925	
16	7.93	5 320	4 050	
18	7.72	5 460	4 190	
20	7.68	5 530	4 260	
22	7.68	5 660	4 390	
24	....	5 590	4 320	

The tests in Table 19 indicate fairly well that it required about 5 or 6 min. for the salt solution to adjust itself to steady conditions after opening the valve. The same conclusion may be drawn from the time required for the decrease in salt rate during the interval from 3-16 to 3-18 to make itself felt in a corresponding decrease of tail-water titration for the interval 3-22 to 3-24. Again, if all the titrations in the fourth column of Table 19 are divided by the salt rate for the interval preceding it by 6 min., a fairly constant ratio results, showing that 6 min. seems to represent the time required for conditions of salt flow

to adjust themselves to changes of a more or less sudden nature. Beginning with the titration, 3 350, for 3-06 P. M., dividing it by the salt rate, 6.41, for 3-00 P. M., two figures of the ratio may be tabulated as follows:

52, 52, 55, 55, 54, 54, 54, 55, 55, 56.

TABLE 20.—MINUTE TAIL-WATER SAMPLES, TEST *U*.

Time. H. M.	Salt rate during preceding interval, in liters per second.	Titration of tail-water sample, in hundredths of cubic centimeters.	Titration, less 1330, for normal head-water, in hundredths of cubic centimeters.	Remarks.
4-35	....	.....	.....	Started minute samples.
36	....	1 345	0	
37	....	1 350	0	
38	....	1 310	0	Started salt solution.
39	?	1 315	0	
40	?	1 430	100	Half-liter samples taken from Pump 25.
41	8	5 300	3 970	
42	8	6 695	5 365	
43	8	6 510	5 180	
44	8	6 485	5 155	

In Test *U*, again, it is seen that it requires 5 or 6 min. for the conditions of flow to become steady. The samples were not continued long enough to make the resulting rate of titration determinable, and the salt rate is uncertain during the first 2 min. of discharge. The final ratio of titration to constant salt rate, carried out to two places, is about 65 (5 160 ÷ 8). This figure could be compared with the ratio, 55, in Test *D*, by taking account of the strength of the salt solution and discharge in the two tests involved. The relative location of points at which samples were taken and the distribution of salt as injected in the head-race are also of moment.

The samples in Table 21 were taken at the end of tests to permit of observations on the weakening concentration when the flow of salt solution is stopped.

In Test 22 it appears that it requires perhaps 8 min. for the salt to disappear completely from the tail-race. The exact instant of closing the salt valve, however, is not recorded, so that there may be an error of a minute or so in the time.

It may be concluded from these results that it requires about 6 or 7 min. to establish uniform conditions of salt flow after changing the

TABLE 21.—TEST 22.—TWO-MINUTE TAIL-WATER SAMPLES.

Time. H. M.	Salt rate during preceding interval, in liters per second.	Titration of tail-water sample of ½ liter.		Titration, less 1310.		Remarks.
		W.	E.	W.	E.	
3-58	7.1	.....	.....	.....	.....	W. samples came from Pump 41 C. L. upper draft-tube.
4-00	?	4 005	4 875	2 695	3 565	
2	0	1 680	1 755	370	445	E. samples came from Pump 42 C. L. of lower draft-tube.
4	0	1 325	1 410	15	100	
6	0	1 325	1 325	15	15	Stopped salt some time after 3-58.
8	0	1 340	1 310	30	00	Started samples 3-58.
10	0	1 310	1 305	00	00	Salt stopped some time between 3-58-30 and 3-59-30.

valve or otherwise altering circumstances in the head-race which produce changes of concentration in the tail-race.

55.—Difficulty in Securing Tail-Race Samples.—It was at first feared that in the tail-race there might be eddies and reverse currents of such a nature as to cause the samples taken from the sampling pumps to be contaminated. It was also feared that there might be reverse currents in the head-race which would allow salt to escape into the head-water of the adjacent units. In either case, the tail-water samples would not be representative.

The first of these sources of error was investigated by shoveling a ton or more of salt into the tail-water at the mouth of the tail-race, some 7 or 8 ft. down stream from the sampling pipes, during which time the sampling pumps were kept in operation. No salt whatever was found in the tail-water samples, thus proving that the water was all flowing outward and that no contamination of samples took place. The actual titrations of ½-liter samples are given in Table 22.

The writer has had experience in tests, however, where the draft-tube and tail-race conditions were not as good as in the case described, the result being that samples of tail-water became instantly contaminated when salt was thrown into the tail-race at a considerable distance down stream from the sampling pipes. In such cases the sampling pipes should be moved back as close to the runner as possible. Even then there will no doubt be occasional cases where it will be found necessary to take samples in the head-race before the feed-water passes through the runner. In this event it will be necessary to take extraordinary precautions to secure a uniform mixture at the distributing system in the head-race, or as quickly after the salt solution is injected into the head-water as possible.

TABLE 22.—TEST 47.—TITRATIONS OF TAIL-WATER BEFORE AND DURING THE INTRODUCTION OF SALT INTO TAIL-WATER 7 OR 8 FT. DOWN STREAM FROM SAMPLING PIPES IN TAIL-RACE.

(Units are $\frac{1}{100}$ c.c. titration of $\frac{1}{2}$ liter of tail-water.)

BEFORE SHOVELING SALT INTO TAIL-WATER.

1330		1314		1320		1330	
	1314		1305		1320		1330
1320		1303		1305		1305	
	1300		1305		1320		1305
1290		1295		1320		1312	
	1295						

DURING THE SHOVELING OF SALT INTO TAIL-WATER.

1305		1300		1323		1310	
	1295		1302		1298		1310
1297		1298		1300		1310	
	1295		1300		1295		1293
1321		1297		1290		1293	
	1300						

MEANS FOR THE FOUR QUARTERS OF TAIL-RACE FROM ABOVE FIGURES.

	Before introducing salt.		During introduction of salt.	
	West.	East.	West.	East.
Upper	1 316	1 315	1 299	1 308
Lower	1 295	1 312	1 303	1 297

Racks Expedite Mixing.—When uniform mixtures must be effected in the head-race as quickly as possible, it will be found that racks placed in the head-race will assist materially. In many cases the distributing pipes may be placed immediately down stream from the usual trash racks, thus making economical use of the power-house installation. However, the openings of the trash racks are entirely too large to raise sufficient head, and it will be necessary, when the trash racks are to be used in this way, to reduce the openings by suitable fillers, or by horizontal strips across the racks uniformly spaced in the vertical direction.

The head which it is necessary to raise to render the velocity uniform in all parts of the section down stream from the racks will depend on local conditions. Indeed, it may be desirable to have this head adjustable, as in the case of a contract specifying the head on the turbines during the tests. In such a case the openings between the strips must be adjustable.

By changing the head lost through the racks, the head on the turbines can be regulated.

The head raised in such cases must be sufficient to render the velocity of the water practically uniform over the entire cross-section of the head-race. This will require a head of not less than 1 ft. on the racks and even more when the velocities in the head-race in its normal condition are very irregular over the cross-section. Racks raising heads of 4 or 5 ft. are not difficult to construct.

Designers of stilling racks have frequently failed to make the openings in the racks small enough. If 1-in. planks are used, it will generally be found that they should be less than 2 in. from center to center. Overloading must be avoided when raising heads on trash racks.

56.—Efficient Mixing, and Design of Sprinklers.—In order to secure this desirable result, it will be necessary to have as many perforations in the distributing sprinklers as economy and operating conditions will permit. Practically, the size of the holes will be limited by the quantity and quality of the water taken up by the centrifugal pump, and the character of the scale which breaks from the distributing pipes themselves. If the pipes are fitted with flushing outlets, say T's, equipped with valves at the middle or end of each sprinkler, any accumulation of particles of scale or other materials which tend to plug the holes might be occasionally flushed out, but even then some of the holes may become clogged and require individual attention. The safe rule to follow is to have a large centrifugal pump, so that the holes may be made both numerous and large, thus not only preventing the clogging of the holes but also improving the mixture by allowing the centrifugal pump to do a great share of the mixing.

E. W. Schoder, Assoc. M. Am. Soc. C. E., of Cornell University, has made some experiments which will assist in designing efficient sprinklers. The results are given in Table 23.

The diameters given in Table 23 refer to the size of the drill. The diameters of the holes were not measured. The holes were drilled in 2-in. black pipe, and no attempt was made to remove burrs on the inside of the pipe.

The discharges given in Table 23 were measured with jets singly in action, with the tank water surface about 22 ft. above the orifices. A measure of the loss of head in the pipe leading from the tank to the orifices is given by the measured discharge with the orifice pipe

removed. With a difference of level of 20.5 ft., the discharge was 55.5 liters per min., the velocity in the 1-in. pipe being about 6 ft. per sec. We may take 20 ft. as the loss of head for this discharge. With the loss taken proportional to the square of the discharge, the ⅜-in. orifice discharge caused 4.6 ft. loss of head, leaving 17.4 ft. as the approximate effective head. The heads given in Table 23 are based on such calculations.

TABLE 23.—CHARACTER OF DISCHARGE FROM HOLES IN 2-INCH PIPE SPACED 2 INCHES APART. FROM EXPERIMENTS BY PROFESSOR E. W. SCHODER.

Nominal size of holes.	Diameter of holes in inches.	Discharge, in liters per minute.	Head, in feet.
5/16 in.	0.312	19.4	19.6
7/32 in.	0.219	10.3	21.5
No. 10	0.194	8.28	22.0
No. 20	0.161	7.46	22.5
No. 30	0.128	4.58	22.5
No. 40	0.098	2.72	22.5
No. 50	0.070	1.31	22.5
No. 60	0.040	0.222	22.5
3/8 in.	0.375	26.6	17.4

A photograph of all the jets except the ⅜-in., discharging simultaneously vertically upward, shows the relative heights reached in air, as well as the orifice pipe arrangement. Another photograph shows the tank, the pipe line, and the orifice pipe resting on the bridge spanning the canal.

For the tests on the jets submerged in flowing water, ½ lb. of Prussian blue was dissolved in about 500 gal. of water. This gave an intensity such that one's finger nail became invisible when submerged 1¼ in. beneath the surface in bright sunlight.

During the tests with the colored water, the velocity of flow in the canal was 1.8 ft. per sec. where the jets issued.

The jets were tested issuing horizontally at right angles to the direction of flow in the canal and also issuing vertically. The pulsations in the canal flow caused a lateral swaying of the colored stream at from about 2 to 4-sec. intervals.

When issuing vertically upward, the 2-in. orifice pipe being perpendicular to the flow in the canal, a sample of water near the surface about 4 ft. down stream from the orifice was caught in a bucket. By noting the intensity of the color of this sample, some indication

of the diffusion was obtained. Owing to the large relative dilution, the method fails for small orifices.

In all cases the color seemed to be uniformly distributed across the band for several feet down stream. Below about 6 ft. down stream, the flow pulsations caused localization in the colored stream in swellings of the colored band and separation into two bands. The general appearance, however, was that of a wavy, slowly widening band of blue, somewhat like smoke from a chimney. Table 24 gives the measurements.

TABLE 24.

Orifice.	Direction.	Submergence, in inches.	Distance down stream from orifice, in inches.	Distance out from plane of orifice of edges of strongly colored stream, in inches.	Instantaneous width of colored stream, in inches.	Color of sample 4 ft. down stream usually from the surface to 6 in. below.
⅝-in....	Horizontal.	11½	2	3 to 9		
			6	5½ to 11½		
			30	4 to 17		
	Vertical...	16	24		6 to 12	Strong at surface.
		15	9	(Boils up to surface.)		
5/16-in...	Horizontal.	11½	2	4 to 6		
			12	6 to 12		
			42	9 to 18		
	Vertical...	16	26		7½	About same color top 6 in. and top 12 in.
			8 ft.	15 to 18		
7/32-in...	Vertical...	16½	26	6		None.
			8 ft.	12 to 15		
		10½				Good.
	Horizontal.	11½	2	4 to 8		
			12	8 to 18		
			42	11 to 21 and 9 to 20	6 to 7	
No. 10...	Horizontal.	6	2	3½ to 7		
			12	7 to 12		
			40	9 to 18	6	
	Vertical....	16	26		5 in. through 8-in. range, none sidewise.	
		10½				Faint.
No. 20...	Horizontal.	6	2	4 to 7½		
			12	5½ to 12½	4	
			38	9 to 17½	5	
No. 30...	Horizontal.	6	2	3 to 6		
			12	5½ to 9	3	
			36	5 to 13	4 to 5	
No. 40...	Horizontal.	6	2	3 to 5		
			8	3½ to 8	2½	
			36	5 to 11	3	
No. 50...	Horizontal.	6	2	2½ to 4½		
			6	3½ to 6		
			32	Middle of faint band about 5 in. out.		
No. 60...	Horizontal.	6	2	1½ to 2		
			4	1½ to 4½	1	

57.—As an illustration, it may be observed that the velocity of the water in the canal as it flowed past the orifices in the pipe was about 1.8 ft. per sec., or approximately what it would be in an ordinary open

head-race. The object is to determine the sizes, spacing of perforations, and spacing of sprinkling pipes, so as to obtain as uniform a mixture as possible at the time of injection, thus assisting the mixing as much as possible at the outset.

Fig. 2 shows an efficient system of holes, based on the results of the experiments just described, but, undoubtedly, this can be improved when more extensive experiments are made.

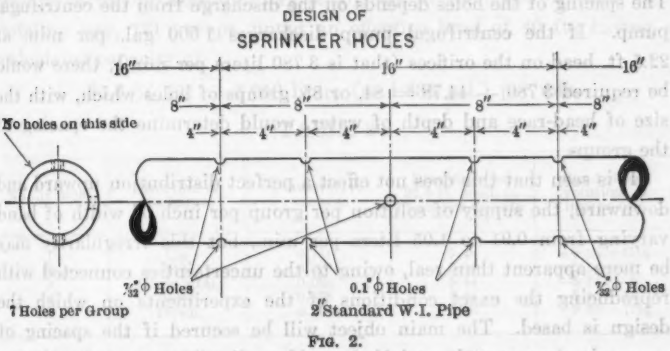


FIG. 2.

The following are the data for a design for sprinkler holes: Intake velocities, 1.8 ft. per sec.; head on sprinkler orifices, 22.5 ft.; discharge of $\frac{7}{32}$-in. orifice, 10.5 liters per min.; discharge of No. 20 drill-hole, 7.46 liters per min.; discharge of No. 40 drill-hole, 2.72 liters per min.; distances out to edges of band supplied with salt by $\frac{7}{32}$-in. hole, 11 and 21 in.; the same for No. 20 hole, 5 and 13 in.; the same for No. 40 hole, 5 and 11 in.

TABLE 25.—DIMENSIONS.

Number of holes.	Diameter of drill, in inches.	Discharge for 22½ ft. head, in liters per second.	Distance to edges of salt band, in inches.	Width of band served, in inches.	Supply of solution per inch of width, in liters per minute.	Direction of discharge of hole.	Position of hole in pipe.
1	7/32	10.5	+11 to +21	10	1.05	Up.	Top.
2	0.098	5.44	+ 5 to +11	6	0.91	Up.	Top.
1	7/32	10.5	− 5 to + 5	10	1.05	Up stream.	Up-stream face.
2	0.098	5.44	− 5 to −11	6	0.91	Down.	Bottom.
1	7/32	10.5	−11 to −21	10	1.05	Down.	Bottom.
Total.....		44.78					

It will be seen from Table 25 that it is intended to keep the holes under a head of 22.5 ft., and that each sprinkler extending across the head-race at right angles to the current is to supply a band of salt 42 in. wide, extending 21 in. above and below the center-line of pipe. (This neglects the diameter of pipe.) Hence the spacing between the sprinklers should be 42 in. This spacing can be altered by changing the pressure on the orifices corresponding to the desired alteration. The spacing of the holes depends on the discharge from the centrifugal pump. If the centrifugal pump discharges 1 000 gal. per min. at 22.5 ft. head on the orifices (that is 3 780 liters per min.), there would be required $3\,780 \div 44.78 = 84$, or 85, groups of holes which, with the size of head-race and depth of water, would determine the spacing of the groups.

It is seen that this does not effect a perfect distribution upward and downward, the supply of solution per group per inch of width of band varying from 0.91 to 1.05 liters per min., but this irregularity may be more apparent than real, owing to the uncertainties connected with reproducing the exact conditions of the experiments on which the design is based. The main object will be secured if the spacing of groups is close enough, and if the orifices distribute the salt solution well over the entire cross-section of the head-race. There is room for more experimentation on this subject, for, in cases where the tail-water samples must be taken in the head-race, thus necessitating the best possible mixture immediately after the injection of the salt solution, it is imperative to have nice work in the design and operation of the distributing system.

The actual design of the sprinkler orifices is shown by Fig. 3. As there was no special difficulty in securing mixtures in these tests, little attention was paid to rigid rules for the design and operation of the sprinkler system. The orifices were usually under a head of about 40 ft., so that it is hard to state from the results of the orifice tests just how thoroughly the sprinkler system effected a mixture independently of the turbines. A better idea on this point may be secured by examining the results of the distribution tests which have been discussed in Section 52 *et seq.*

58.—Discharge of the Orifices in Tests.—It was intended to follow the general method just discussed for the design of the sprinklers in the tests, but it is believed that the holes were not exactly of the sizes

originally contemplated. Moreover, more or less loose scale in the pipes tended to stop up the holes, thus interfering at times with a full realization of the discharges anticipated. The approximate results for one group of holes are shown in Table 26, from which it will be seen that the total discharge of the eight orifices, under 22.5 ft. head, should be 31.24 liters per min., or about $31.24 \times \sqrt{40 \div 22.5} = 41.6$ liters per min. per group of holes under 40 ft. head. There were 120 groups, with one hole of extra small size at each end of each pipe, totaling, say, 121 groups, under an effective head of 40 ft., having a calculated capacity of

$$41.6 \times 121 = 5\,050 \text{ liters (or } 1\,335 \text{ gal.) per min.}$$

TABLE 26.

No. of holes.	Direction of discharge.	Diameter, in inches.	Discharge for 22.5 ft. head, in liters per minute.	Band of salt solution, in inches out.
1	Up.	0.161	7.46	+9 to +17½
2	Up.	0.098	5.44	+5 to +11
2	Horizontal.	0.098	5.44	−5 to + 5
2	Down.	0.098	5.44	−5 to −11
1	Down.	0.161	7.46	−9 to −17½
Total.....			31.24	

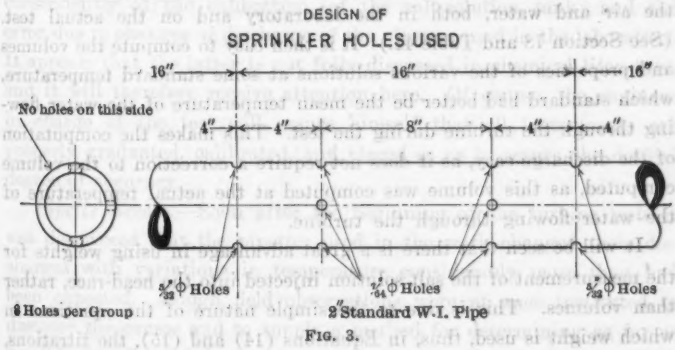


FIG. 3.

The holes, apparently, were of a somewhat different diameter than stated above, but the fact that the discharge of the centrifugal pump, as given in Section 51, and therefore of the sprinkler system, was always less than 1 000 gal. per min., would indicate that a considerable portion of the area of the holes was plugged in the majority of cases.

59.—Leaking Pipes.—Another difficulty experienced in getting representative samples of tail-water was caused by leaks in the ¼-in. risers leading up from the tail-water. The crown of the tail-race arch was from 6 to 8 ft. under water, causing a vertical eddy flowing toward the power-house and against the face of the tail-race wall. This eddy necessarily carried a large portion of tail-water from the adjacent units and river, thus diluting the water next to the power-house wall for a depth of at least 6 ft. As the ¼-in. tail-sample pipes passed up through this water in the early tests, several of the pipes which became leaky on one or two occasions caused a contamination of the samples, which necessitated rejecting the corresponding titrations at such times.

After Test 23, however, the tail-race sampling pipes were moved back into the draft-tube, so that all remaining tests were free from this trouble, the pipes being entirely immersed in the tail-water from the turbine being tested.

60.—Error 7.—Errors Due to Variations of Temperature.—If the temperature varies during a test or series of tests, it will cause changes in the capacities of the measuring instruments, in the properties of the solutions, and in the determinations of the end points of titrations. Hence, it is necessary, when volumetric measurement of samples is used, to observe the temperatures of all the solutions, and of the air and water, both in the laboratory and on the actual test. (See Section 78 and Table 41.) It is then easy to compute the volumes and properties of the various solutions at some standard temperature, which standard had better be the mean temperature of the water flowing through the turbine during the test. This makes the computation of the discharge easy, as it does not require a correction to the volume computed, as this volume was computed at the actual temperature of the water flowing through the turbine.

It will be seen that there is a great advantage in using weights for the measurement of the salt solution injected into the head-race, rather than volumes. This is due to the simple nature of the equations in which weight is used, thus, in Equations (14) and (15), the titrations, p, p_1, p_2, and the rate of weight, w, of salt solution discharged into the head-race, are all that is necessary to know in order to determine W_2, the weight of the water discharged by the turbine. Again, it is this weight which is required to determine the theoretical power with which the turbine is to be charged in computing the efficiency. These equations

are nearly independent of temperature, as the percentages, p, p_1, and p_2, determined by titration, are the same at all temperatures.

For these reasons, the method of weights involved in Equations (5) to (28), inclusive, is recommended as being better suited to turbine testing than the method of volumes used in the later equations. This does not mean that the silver nitrate is weighed. Indeed, the chemical determination is still volumetric, in that the titrations are conducted in exactly the same manner as before, the only difference being that the sizes of samples are determined by weighings.

Finally, any changes in the end-point, or strength, of the silver nitrate, due to changes in temperature during treatment in the laboratory, will be eliminated by the method of special dilutions and group titrations explained in connection with Errors 1 and 2, Sections 46, 47, and 48.

61.—Error 8.—Instrumental Errors.—It is scarcely necessary to enter at length into a discussion of the errors and their net effect which constitute Error 8. There are two errors of this nature, however, to which great attention must be given by the engineer in charge of the test, and these two only will be discussed somewhat in detail. These are the error likely to occur by reason of a change in the capacity, and consequently of the calibration, of the salt-solution tank; and the error due to changes of discharge of the pipettes used in the laboratory. It appears that the latter is not fully discussed in chemical literature, and it will therefore receive attention here. Of course, the engineer in charge of the test will assure himself that all instruments are properly graduated, calibrated, and placed so as to secure the desired degree of accuracy.

Pipette Errors.—Soon after the beginning of the turbine tests it was discovered that the pipettes used in the work changed their discharges with variations in temperature considerably more than had been expected. Rough field observations were at once instituted to discover the causes and to supply a method for determining as far as possible the effect of changes in temperature on their discharge. These tests showed at once that the discharge changed by several times the change in volume of a 10-c.c. pipette, due to a given change in temperature in the neighborhood of 20° cent. After due reflection it was concluded that the change in viscosity, surface tension, cohesion, etc., had a preponderating influence, when the temperature changed,

so as to augment the change in discharge far beyond what the mere change in volumetric capacity of the pipette would account for.

Moreover, it was found, as had been expected, that the concentration of salt in solution had a relatively large influence on the discharge of a pipette, and it became necessary to investigate this source of error, as well as the former. Accordingly, the pipettes were sent to the U. S. Bureau of Standards, Washington, after the turbine tests had been concluded, with the request that they be calibrated with distilled water and salt solutions of 150, 250, and 300 grammes concentration per liter. To ascertain the effect of temperature, each pipette was to be calibrated at three different temperatures, 15°, 20°, and 25°, cent., for its discharge of each of the four solutions specified. By these calibrations it was hoped to discover the law of variation.

Actual calibrations of the pipettes were made by the Bureau only at 20° cent., and the capacities at 15° and 25° cent. were then computed on the basis that the increment of discharge was equal to the increment of volumetric capacity, which was the very thing these desired calibrations sought to avoid. However, it was finally decided to make practical calibrations at the Carnegie Institute of Technology, and these were carried out with considerable success, by actually comparing the volume discharged by a pipette with a fixed volume.

The method of conducting one of these tests was, first, to find a place where a fairly constant temperature could be secured, and second, to discharge the pipette into a burette, the capacity of which had been previously ascertained by calibration. The solution used was made up to a known concentration of salt, except where it was required to test the discharge of distilled water. By making a number of such tests at various temperatures with each solution, it was possible to determine the effects of changes in temperature and concentration.

Fig. 4 shows the results of these tests in diagrammatic form, and Table 27 gives the actual observations. The use of the diagrams will be readily understood, as the discharge of any given pipette can be found at a point on the corresponding diagram opposite the known temperature and vertically above the concentration of the salt solution used, distilled water, of course, being indicated by zero concentration.

62.—*Calibration of Salt-Solution Tank.*—In order to secure the desired degree of accuracy in measuring the salt solution consumed during a test, the solution tank must be so high that a vertical gauge-

board, of sufficient length, can be attached to the side, to admit of readings of the level of the solution in the tank with the necessary precision.

As it will ultimately be required to know the time rate at which the turbine develops power, it will also be necessary to note the time during which the surface of the salt solution falls from one level to another.

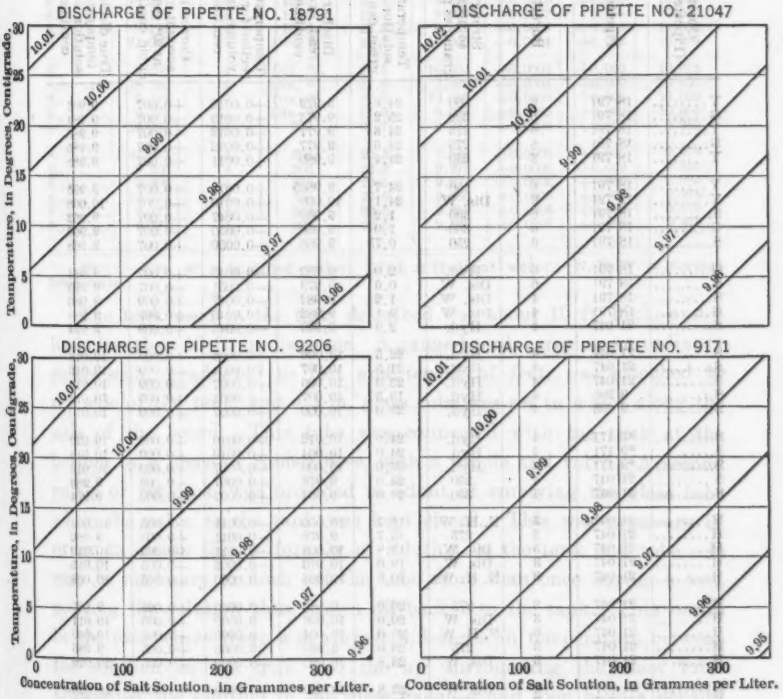


FIG. 4.

This time interval must be determined with the same precision as the quantity of salt solution consumed. Hence, it will be the duty of the engineer to determine the size of the tank and the length of the test, and these two elements will depend on the accuracy with which the timepiece and the tank-gauge can be read, and on the predetermined limit of permissible error.

TABLE 27.—Pipette Calibrations Corrected.

Taking Burette No. 6 = 100.070 c.c. at 20° cent.
" No. 4 = 100.090 c.c. at 20° cent.
" No. 3 = 100.05 c.c. at 20° cent.

Observer (Pipetted by).	Pipette No.	Burette No.	Strength of solution, in grammes per liter.	Temperature of solution, in degrees, centigrade.	Discharge as read, in cubic centimeters.	Temperature correction for burette volume, in cubic centimeters.	Correction on account of error, at 20° cent., in cubic centimeters.	True discharge at temperature of solution, in cubic centimeters.
Y.....	18 791	6	300	24.6	9.973	+0.0012	+0.007	9.981
G.....	18 791	6	300	25.2	9.973	+0.0013	+0.007	9.981
Y.....	18 791	6	275	24.6	9.977	+0.0012	+0.007	9.985
G.....	18 791	6	275	24.6	9.977	+0.0012	+0.007	9.985
Y.....	18 791	6	250	24.4	9.980	+0.0011	+0.007	9.988
Y.....	18 791	6	150	24.7	9.9865	+0.0012	+0.007	9.995
Y.....	18 791	6	Dis. W.	24.1	10.000	+0.0010	+0.007	10.008
S.....	18 791	6	300	1.2	9.960	−0.0047	+0.007	9.962
G.....	18 791	6	300	1.0	9.958	−0.0050	+0.007	9.960
S.....	18 791	6	250	0.75	9.966	−0.0050	+0.007	9.968
G.....	18 791	6	Hyd.	2.0	9.982	−0.0045	+0.007	9.984
S.....	18 791	6	Dis. W.	0.6	9.973	−0.0051	+0.007	9.975
S.....	18 791	4	Dis. W.	1.2	9.982	−0.0047	+0.009	9.986
G.....	18 791	4	Dis. W.	2 1	9.982	−0.0045	+0.009	9.986
S.....	21 047	4	Hyd.	2.0	9.990	−0.0045	+0.009	9.994
S.....	21 047	4	Hyd.	22.5	10.006	+0.0006	+0.009	10.016
S.....	21 047	4	Hyd.	21.5	10.007	+0.0004	+0.009	10.016
S.....	21 047	4	Hyd.	23.0	10.006	+0.0007	+0.009	10.016
S.....	9 206	4	Hyd.	19.5	10.007	−0.0001	+0.009	10.016
S.....	9 206	4	Hyd.	23.0	10.000	+0.0007	+0.009	10.010
S.....	*9 171	4	Hyd.	24.0	10.012	+0.0010	+0.009	10.022
S.....	*9 171	4	Hyd.	24.0	10.004	+0.0010	+0.009	10.014
S.....	*9 171	4	Hyd.	23.0	10.000	+0.0007	+0.009	10.010
S.....	21 047	4	250	23.0	9.976	+0.0007	+0.009	9.986
S.....	21 047	4	300	23.0	9.964	+0.0007	+0.009	9.974
H.....	21 047	3	N. H. W.	15.7	10.000	−0.0011	+0.005	10.004
H.....	21 047	3	275	15.7	9.978	−0.0011	+0.005	9.982
H.....	21 047	3	Dis. W.	15.8	10.008	−0.0011	+0.005	10.012
H.....	21 047	3	Dis. W.	19.0	10.010	−0.0002	+0.005	10.015
H.....	21 047	3	N. H. W.	20.0	10.004	0.0000	+0.005	10.009
H.....	21 047	3	275	20.0	9.980	0.0000	+0.005	9.985
H.....	21 047	3	Dis. W.	20.0	10.006	0.0000	+0.005	10 011
H.....	21 047	3	N. H. W.	20.0	10.004	0.0000	+0.005	10.009
H.....	21 047	3	275	20.0	9.980	0.0000	+0.005	9.985
H.....	21 047	3	Dis. W.	23.8	10.020	+0.0009	+0.005	10.026
H.....	21 047	3	Dis. W.	23.8	10.020	+0.0009	+0.005	10.026
H.....	21 047	3	275	23.8	9.980	+0.0009	+0.005	9.986
H.....	21 047	3	275	23.8	9.980	+0.0009	+0.005	9.986
H.....	21 047	3	275	25.0	9.980	+0.0012	+0.005	9.986
H.....	21 047	3	Dis. W.	25.0	10.022	+0.0012	+0.005	10.028
H.....	21 047	3	N. H. W.	25.0	10.006	+0.0012	+0.005	10.012
S.....	21 047	4	Hyd.	22.0	10.000	+0.0005	+0.009	10.010
S.....	21 047	4	Hyd.	22.5	10.000	+0.0006	+0.009	10.010
S.....	9 206	4	Hyd.	23.0	10.002	+0.0007	+0.009	10.012
S.....	9 206	4	Hyd.	23.0	10.002	+0.0007	+0.009	10.012

Note.—Dis. W. = Distilled water. Hyd. = Hydrant water. N. H. W. = Normal head-water.

TABLE 27.—(*Continued.*)

Observer (Pipetted by).	Pipette No.	Burette No.	Strength of solution, in grammes per liter.	Temperature of solution, in degrees, centigrade.	Discharge as read, in cubic centimeters.	Temperature correction for burette volume, in cubic centimeters.	Correction on account of error at 20° cent., in cubic centimeters.	True discharge at temperature of solution, in cubic centimeters.
S.....	18 791	4	Hyd.	23.0	9.998	+0.0007	+0.009	10.008
S.....	18 791	4	Hyd.	24.0	9.998	+0.0010	+0.009	10.008
S.....	9 171	4	Hyd.	23.0	9.990	+0.0007	+0.009	10.000
S.....	9 171	4	Hyd.	23.5	9.996	+0.0008	+0.009	10.006
S.....	21 047	4	Dis. W.	21.7	10.000	+0.0004	+0.009	10.009
S.....	9 206	4	Dis. W.	21.5	10.004	+0.0004	+0.009	10.013
S.....	18 791	4	Dis. W.	21.8	9.998	+0.0004	+0.009	10.007
S.....	9 171	4	Dis. W.	21.3	10.000	+0.0003	+0.009	10.009
S.....	9 171	4	Dis. W.	21.9	9.994	+0.0005	+0.009	10.004
S.....	21 047	4	300	21.7	9.974	+0.0004	+0.009	9.983
S.....	21 047	4	300	22.3	9.968	+0.0006	+0.009	9.978
S.....	9 206	4	300	22.5	9.976	+0.0006	+0.009	9.986
S.....	9 206	4	300	22.5	9.974	+0.0006	+0.009	9.984
S.....	18 791	4	300	22.6	9.968	+0.0006	+0.009	9.978
S.....	9 171	4	300	22.6	9.968	+0.0006	+0.009	9.978
S.....	9 171	4	300	22.5	9.966	+0.0006	+0.009	9.976

NOTE.—Dis. W. = Distilled water. Hyd. = Hydrant water. N. H. W. = Normal head-water.

The tank used in the tests described was about 10 ft. high, and a little less than 10 ft. in diameter. A gauge-board, carrying an ordinary gauge-scale, graduated to feet and tenths of feet, was attached to the side of the tank, and a 1-in. glass tube was set in a slot along the axis of the board. This tube was connected with the tank at the bottom by a piece of rubber hose, with a nipple and valve. A second valve, or pet-cock, was provided to admit of emptying the glass tube while the valve to the tank was kept closed. This was necessary in order to insure the uniformity of solution in the tank and tube. It may be necessary to flush out the tube more than once during a test, pouring the solution thus drawn off back into the tank. This would be the case where there was a great difference in temperature between the solution in the tank and the air surrounding the glass tube. Otherwise, the solution in the glass gauge would soon have a density different from that in the tank, thus indicating false surface levels.

On the face of the board, opposite the edge graduated in feet and tenths of feet, a calibration scale was marked off with a lead pencil and carpenter's square. This calibration was made by filling the tank with water and then drawing out equal volumes successively, making a straight horizontal calibration mark after each draft of the standard

volume. This standard volume may be constructed with a piece of 12-in. pipe, or the volume may be determined from a weighing, correcting for temperature variations, in which case an investigation must be made to ascertain whether the water used has any serious deviation in its density from that of distilled water. This latter statement, however, is made only when considering the calibrator as representing a true unit of volume. It is not necessary, for the purpose of efficiency tests, to have the tank or calibrator calibrated to deliver true volumes by weighings of distilled water. The actual feed-water may be used in place of distilled water, any error made in assuming its density to be that of distilled water being compensated by the counter error committed when the theoretical power of the feed-water is calculated on the basis that it is distilled water. Thus, the product of the observed volume, too large in a certain ratio, and the density of distilled water, too small in a like ratio, occur in the formula for efficiency. Therefore, the efficiency is correctly determined, although the discharge derived from the tank observations may be slightly in error. This question is more fully discussed under Error 10, Section 73.

63.—When Does a Test Begin and End?—In operating this tank for a test, the regulating valve leading to the distributing pipes was opened about 7 or 8 min. before the signal for the beginning of the test, which was the ringing of all the bells on the telephone system leading to the different observers. The valve-man then quickly adjusted the valve, by direction of the gauge-reader, until the meniscus of the solution descended in the glass tube at a specified rate, usually about 30 or 40 sec. per calibration division, which corresponds to about 8.46 cu. ft. This usually consumed 2 or 3 min. After this, the valve was adjusted from time to time during the tests so as to maintain, approximately, a uniform rate of discharge.

The gauge-reader then began to note the instants of time when the meniscus passed the successive calibration divisions, the data being entered on the second sheet. One minute before the beginning of the test a preliminary signal was given, and at the beginning there was another signal. The gauge-reader, however, merely draws a line under the last recorded item at the beginning signal, making no other observations. He continues to enter the successive transits of the calibration marks until the signal at the end of the test is sounded,

when he again draws a line under the last recorded item. After taking several more transits, for 1 or 2 min., the flow of salt solution is stopped and the test is at an end.

The adequacy of this proceeding will be seen by noting that there is really no well-defined beginning or end of a turbine test, and that, in fact, the rate of flow of salt solution is substantially the same whether it is determined from the calibration divisions immediately preceding or following the lines drawn when the signals are given for the nominal beginning and end of the test.

The only serious care necessary to take is to see that the rate of flow from the tank is about the same for the first few divisions before and after the beginning, and before and after the end, of the test, so that the time of discharging the standard volume is nearly the same at the first and last divisions, which are indicated on the record by lines drawn under the entries made by the gauge-reader.

If stop-watches are used it will be necessary to run the test for 15 min. in order to limit errors to $\frac{1}{10}$ of 1% with certainty, because, for ordinary field methods, there would be likely to be an error of nearly 1 sec. Stop-watches are not very satisfactory for accurate time observations.

If a chronograph is used, time can be determined with a precision of $\frac{1}{10}$ of 1%, with ordinary care, in 2 or 3 min., or even less. Hence it is clear that, with a given size of tank, a saving of chemical can be effected by the use of a chronograph, where high precision is demanded, as was the case in the tests described herein.

64.—Gravimetric Calibration of Calibrator.—Fig. 5 exhibits the results of a series of calibrations of the iron pipette, or calibrator, which was used in graduating the capacity scale for the gauge-board at the side of the salt-solution tank, as described in Section 62.

It will be seen from Fig. 5 that the three calibrations of the calibrator were made at different times, with a change of observers after the first series. The volumes at 20° cent. computed from the averages of the weighings in the three series are, respectively:

July 13th, 1914	8.4661 cu. ft.
October 22d, 1914	8.4662 cu. ft.
October 27th, 1914	8.4651 cu. ft.

It is possible that this close agreement is principally due to the fact that the weighings were all made on the same scale, which, though for commercial purposes, was carefully adjusted with standard weights

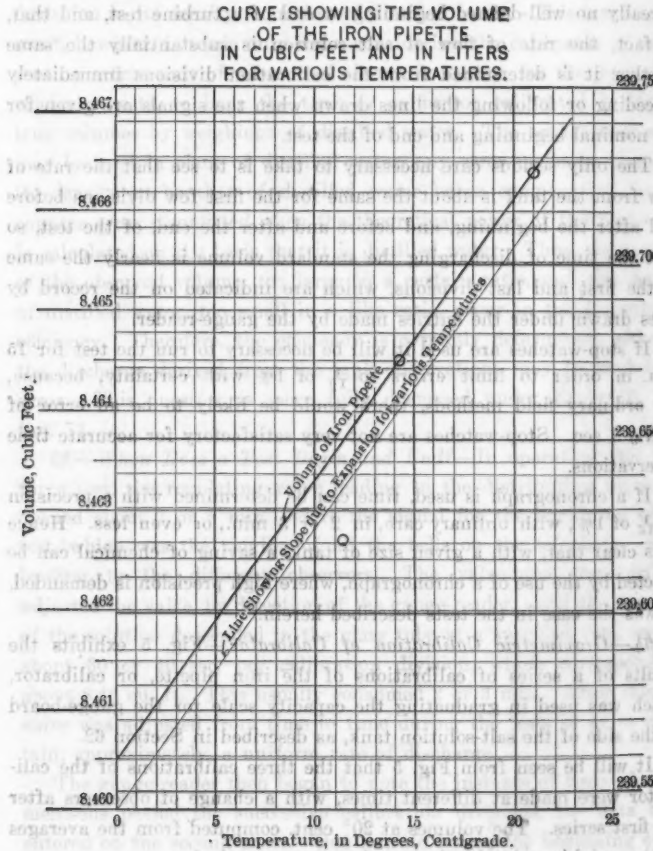


FIG. 5.

prior to each series of weighings. There is no reason to doubt the high precision of these weighings, considered as the results of engineering operations in the field.

TABLE 28.—Calibration of Iron Pipette by Gravimetric Method.

By G. H. Bancroft and R. E. Ward.

July 13th, 1914. Howe Scale No. 656 849.

No.	Weight, in Pounds.			Temperature of water, in degrees, centigrade.	Time.	Remarks.
	Tank full.	Tank empty.	Weight of water.			
*1.....	1 606.75	1 078.00	528.75	21.0	A. M.	Canal water was used.
*2.....	1 605.28	1 078.12	527.16	20.8	A. M.	
3.....	1 605.75	1 078.37	527.38	20.5	A. M.	
4.....	1 605.75	1 078.25	527.50	21.3	P. M.	
5.....	1 605.55	1 078.10	527.45	21.0	P. M.	
6.....	1 605.40	1 077.87	527.53	21.1	P. M.	
7.....	1 605.40	1 078.00	527.40	21.1	P. M.	
8.....	1 605.20	1 077.67	527.53	21.0	P. M.	
Mean..	1 605.508	1 078.043	527.465	21.0		

*Readings 1 and 2 omitted in average.

Weight of 1 c.c. of water at 4° cent. = 1.0 gramme. (From Landolt and Börnstein.)
Weight of 1 cu. m. (= 1 000 liters) of water at 4° cent. = 1 000 kg.
1 cu. m. = 35.31658 cu. ft. (Van Nostrand's Chemical Annual.)
1 kg. = 2.20462 lb. (Van Nostrand's Chemical Annual.)

Therefore, weight of 1 cu. ft. of water at 4° cent. $= \frac{1\,000}{35.31658} = 28.31531$ kg.

or weight of 1 cu. ft. of water at 4° cent. $= 28.31531 \times 2.20462 = 62.4245$ lb.
Weight of 1 c.c. of water at 21° cent. = 0.998019 gramme. (Landolt and Börnstein.)
or weight of 1 cu. ft. of water at 21° cent. $= 62.4245 \times 0.998019 = 62.301$ lb.

Therefore, number of cubic feet in iron pipette at 21° cent. $= \frac{527.465}{62.301} = 8.4664$.

1 cu. ft. = 28.31531 liters.
$8.4664 \times 28.315 = 239.73$ liters in pipette at 21° cent.
Linear coefficient of expansion of iron = 0.0000111 for 1° cent. (Landolt).
Cubical coefficient of expansion of iron = 0.0000333 for 1° cent.
To find volume, in cubic feet, at 20° cent.
Difference in temperature = 21° − 20° = 1°.
$1 \times 0.0000333 = 0.0000333$ cubical coefficient of expansion for 1° cent.
Therefore, volume at 20° $= 8.4664 - (8.4664 \times 0.0000333) = 8.4661$ cu. ft.

In order to facilitate the determination of the volume of the calibrator, or large iron pipette, at various temperatures, the curve (Fig. 5) was prepared, and needs no further explanation.

For the purposes of the tests, the volume of this calibrator will be taken from the curve.

65.—Contents and Delivery of Iron Calibrator.—This large calibrator, or iron pipette, Fig. 21, was constructed on the same principle as the pipettes of the chemists. It consisted of a length of 12-in. pipe, in a vertical position, with a conical top and bottom, so as to admit of as complete a discharge from the bottom as possible while any air within would escape at the top. The top of the upper cone was continued upward with a piece of 2-in. pipe for 18 or 20 in., carrying an ordinary water glass at the side on which was a mark indicating

TABLE 29.—Calibration of Iron Pipette by Gravimetric Method. By J. B. Campbell, C. M. Scudder, and D. C. Walser.

October 22d, 1914. Howe Scale No. 656 849.

No.	Weight, in Pounds.			Temperature of Water, in Degrees, Centigrade.				Time.	Remarks.
	Tank full.	Tank empty.	Water.	From hose.	Tank one-half full.	Tank full, Top.	Tank full, Bottom.		
1	.....	*1 139.3	.....	...	....	....	....	A. M.	Tank empty
2	1 668.1	1 140.2	527.9	14.3	14.0	14.0	14.0	8:30– 9:30	but dry.
3	1 668.0	1 139.9	528.1	14.0	14.0	14.0	14.0	9:30–10:25	Canal water
4	1 667.7	1 139.7	528.0	15.2	14.2	14.0	14.0	10:25–11:00	was used.
Mean	1 667.93	1 139.93	528.0						Room temperature 18.0° cent.
				Mean temperature 14.14° cent.					

* Reading 1 omitted in average.

Weight of 1 cu. ft. of water at 4° cent. is 62.4245 lb
Weight of 1 c.c. of water at 14.14° cent. = 0.9992514 gramme. (Landolt and Börnstein.)
Weight of 1 cu. ft. of water at 14.14° cent. = 62.4245 × 0.9992514 = 62.378 lb.

Therefore, number of cubic feet in iron pipette at 14.14° cent. = $\frac{528.0}{62.378} = 8.4645$.

Number of liters in iron pipette at 14.14° cent. = 8.4645 × 28.315 = 239.67.
To find volume of iron pipette at 20° cent. in cubic feet.
Change of temperature = 20° — 14.14° cent. = 5.86° cent.
Cubical coefficient of expansion for 1° cent. = 0.0000333.
Cubical coefficient of expansion for 5.86° cent. = 0.000195.
Therefore, volume at 20° = 8.4645 + (8.4645 × 0.000195) = 8.4662 cu. ft.

the proper level of the water for a standard volume. The capacity, when filled to this mark, was a little more than 241 liters, and the discharge from it when filled with canal water was very close to 240 liters, or about 1 liter remained in the calibrator when it was emptied through the conical bottom which was provided with a throttle valve.

The large iron calibrator had the characteristic of a chemical flask, or chemical pipette, in that the content was greater than the delivery. Owing to this difference, the chemical calibration of the contents does not equal the gravimetric calibration of the delivery, but it will be seen that the capacities of the large solution tank, determined chemically and gravimetrically, agree closely, so that by means of the agreement between the results of the two kinds of calibration of the solution tank, the gravimetric and chemical calibrations of the calibrator check each other very satisfactorily, in view of the unfavorable conditions under which the chemical calibrations of both took place. This calibrator, or iron pipette, is more fully described in Part III.

TABLE 30.—Calibration of Iron Pipette by Gravimetric Method.

By J. B. Campbell, C. M. Scudder, and D. C. Walser.

October 27th, 1914. Howe Scale No. 656 849.

No.	Weight, in Pounds.			Temperature of Water, in Degrees, Centigrade.				Time.	Remarks.
	Tank full.	Tank empty.	Water.	From hose.	Tank one-half full.	Tank full.			
						Top.	Bottom.		
								A. M.	
1	.....	*1 127.8	.....	....	.....	....	....	.....	Tank empty but dry.
2	1 656.9	1 128.8	528.1	11.1	10.0	11.3	9.5	9:55–10:40	3.4° cent. temperature of air.
3	1 656.9	1 129.0	527.9	11.1	10.0	11.2	10.4	10:40–11:17	4.0° cent. temperature of air.
4	1 656.9	1 129.0	527.9	11.1	11.0	11.1	11.0	11:17–11:50	
								P. M.	
5	1 656.9	1 128.6	528.3	11.1	10.5	12.5	10.0	1:40 – 2:20	4.8° cent. temperature of air.
6	1 657.0	1 128.9	528.1	12.5	12.5	12.5	12.2	2:20 – 3:00	6.2° cent. temperature of air.
7	1 657.0	1 129.0	528.0	12.0	12.0	12.0	11.0	3:00 – 3:35	
8	1 657.4	1 129.3	528.1	11.8	12.0	12.0	12.0	3:37 – 4:15	7.6° cent. temperature of air.
9	1 657.2	1 129.1	528.1	....	12.0	12.0	11.6	4:15 – 4:45	
Mean	1 657.025	1 128.9025	528.0625	11.53	11.25	11.83	10.96		Canal water was used.
				Mean temperature = 11.4° cent.					

*Reading 1 omitted in average.

Weight of 1 cu. ft. of water at 4° cent. is 62.4245 lb.
Weight of 1 c.c. of water at 11.4° cent. = 0.999591 gramme. (Landolt and Börnstein.)
Weight of 1 cu. ft. of water at 11.4° cent. = $62.4245 \times 0.999591 = 62.399$ lb.

Therefore, number of cubic feet in iron pipette at 11.4° cent. $= \frac{528.0625}{62.399} = 8.4627$.

Number of liters in iron pipette at 11.4° cent. $= 8.4627 \times 28.315 = 239.62$.
To find volume of iron pipette at 20° cent. in cubic feet.
Change of temperature $= 20° - 11.4° = 8.6°$ cent.
Cubical coefficient of expansion for 1° cent. change = 0.0000333.
Cubical coefficient of expansion for 8.6° cent. = 0.000286.
Therefore, volume at 20° cent. $= 8.4627 + (8.4627 \times 0.000286) = 8.4651$ cu. ft.

66.—Chemical Determination of Capacities.—The chemical calibrations of the solution tank and the iron calibrator are given in Tables 32-38 and 41, Sections 66, 67, and 68. In order to illustrate the methods more fully, the titrations of all the samples in the case of the calibrator are given in Tables 32 and 33. These tables also show the relative accuracy of the chemical work. In particular, attention must be directed to the fact that two of these calibrations are less trustworthy than the remainder, and that all were executed under more adverse conditions than the regular tests of the turbines. Those in October were mostly performed in a drizzling rain, at the close of a long series of regular tests, and after some of the chemists had returned to their duties at various colleges. Those of November were performed by an entirely different corps of chemists, and during very cold weather, with the atmospheric temperature in the neighborhood of zero, Fahrenheit. The writer supervised the measurement of the 21½ liters of salt solution for the lower level of the tank on November 24th, when its temperature was found to be — 2° cent. This was done in the laboratory where the temperature was nearly 20° cent. higher. If such unfavorable conditions could have been avoided, there is no doubt that a precision surpassing that of the best of the turbine tests would have been secured. As the tests stand, however, they are probably more valuable to the engineer as examples of what can be accomplished under adverse conditions, such as are experienced in the field, than they would have been under more perfect laboratory conditions.

It has been stated that two of the chemical calibrations are not as trustworthy as the others. These are the tests of November 23d, at the upper level of the solution tank, and those of November 25th on the iron calibrator. The first of these was rendered untrustworthy by a failure to measure accurately the quantity of salt solution introduced into the tank from the calibrator. There were, however, nine samples, averaging about 200 c.c. each, taken for this purpose, so that 1 800 c.c. have been deducted from the capacity of the calibrator to ascertain the value of q for this test. In the case of the calibration of the iron calibrator on November 25th, the laboratory work does not check with sufficient exactness to support confidence in the result, as shown by the calculations in Table 31.

The titrations for the upper and lower levels of the large tank show that there was relatively considerable salt still remaining in the tank

after it had been thoroughly cleaned and prepared for the chemical calibration, and that this fact is more apparent at the lower level, as might be expected.

TABLE 31.—CALCULATIONS OF CONCENTRATION OF NORMAL HEAD-WATER SAMPLES TAKEN FROM LARGE TANK AND CALIBRATOR BEFORE INTRODUCING SALT SOLUTION IN THE CHEMICAL CALIBRATIONS.

Symbol.	Calibrator.		Tank.			
			Upper level.		Lower level.	
	October 14th.	November 25th.	October 15th.	November 23d.	October 16th.	November 24th.
R'	100.65	*100.97	100.48	100.59	100.51	100.64
c'_2	5 490.9	5 326.9	5 757.1	5 454.0	5 513.6	5 899.5
c	550 295	535 030	558 091	530 851	523 936	533 814
r'	99.68	100.00	99.51	99.61	99.53	99.66
$R\,c'_2$	552 659	587 857	578 473	548 618	554 172	593 726
c_1	23.72	28.27	204.82	178.37	303.79	601.16

*Interpolated value from following calculations.

By Equation (49), Section 21, the titration of normal head-water, November 25th, is found, by using the data of Table 34 (Plate XXXVI), to be

$$v_1 = \frac{100.82 \times 5\,326.9 - 535\,030}{99.85} = 20.31,$$

whereas the result should have been about 28 c.c., as may be seen by examining the titrations of the normal head-water on the turbine tests, either observed or computed, see Part IV, or Table 7, Section 46, where it will be found that the titration of the normal head-water was about 26 c.c., as an average, when the strength of the silver nitrate was 1.562, and about 28 c.c. when the strength was 1.45, which is in perfect accord with theory, as

$$\frac{1.562}{1.45} \times 26 = 28.$$

Notice that v_1 for the test of October 14th may be computed to be 23.7 c.c., which is within about 2 c.c. of what it should be, and thus sufficiently accurate.

In order to correct this error for November 25th, slight as the effect is, the value of R' has been computed by using Equation (99), substituting about 28 c.c. for c_1, so that

$$R' = \frac{535\,000}{5\,299} = 100.96$$

Accordingly, this value (more correctly computed = 100.97) of R' has been arbitrarily substituted at the proper place in Table 35, thus giving a somewhat better estimate of the volume. The result, however, is still somewhat doubtful, as the samples of salt solution and other liquids involved do not run as uniformly as they should. Individual titrations of the same solution should agree within about 0.1 c.c., and averages of from five to ten such titrations should check similar averages within $\frac{1}{20}$ of 1% when the total titration of any sample is about 50 c.c. Thus the chemical work of November, as a whole, will not be found so good as that of the earlier tests. These errors, however, are more accidental than systematic, so that curves plotted from numerous tests should not be seriously in error, theoretically being of the same precision as the earlier curves, if there are correspondingly more observations.

To gain a better view of the chemical work, the titrations of the chemical gaugings of the capacity of the iron calibrator are given in Tables 32 and 33.

TABLE 32.—TITRATIONS FOR CHEMICAL DETERMINATION OF VOLUME OF IRON CALIBRATOR.

OCTOBER 14TH, 1914.

	Salt solution. 1/100 c.c.		Special dilutions. 1/100 c.c.		Tail-water. 1/100 c.c.	
	5 485	5 480	.....	.....	5 500	5 515
	5 475	5 470	5 500	5 500	5 510	5 500
	5 460	5 458	5 500	5 495	5 505	5 505
	5 480	5 484	.....	.....	.....	.....
	5 480	5 477	.....	.....	.....	.....
Means.....	5 476	5 473.8	5 500	5 497.5	5 505	5 506.7
Average.....	5 474.9		5 498.8		5 505.8	

OCTOBER 15TH, 1914.

	5 470	5 480	5 490	5 485	5 495	5 495
	5 470	5 460	5 485	5 490	5 495	5 495
	5 465	5 460	5 485	5 490	.....	.....
Means.....	5 468.3	5 466.7	5 486.7	5 488.3	5 495	5 495
Average.....	5 467.5		5 487.5		5 495.0	
Final average....	5 471.2		5 493.1		5 500.4	

TABLE 33.—TITRATIONS FOR CHEMICAL DETERMINATION OF VOLUME OF IRON CALIBRATOR, NOVEMBER 25TH, 1914.

Titrated November 26th, 1914.

Salt solution, 1/100 c.c.			Special dilutions, 1/100 c.c.			Tail-water, 1/100 c.c.		
1	5 292	5 300	1	5 300	5 306	*B.*	(5 365)	(5 372)
2	5 325	5 325	2	5 332	5 332	*T.*	(5 305)	(5 310)
3	5 272	5 276	3	5 290	5 295	*B.*	(5 359)	(5 360)
4	5 326	5 317	4	5 340	5 344	*T.*	5 335	5 333
			3	(5 260)	(5 263)	*B.*	5 339	5 323
			4	5 343	5 345	*T.*	5 337	5 335
Means..	5 303.8	5 304.5		5 321	5 324.4		5 337	5 330.3
Average	5 304.1			5 322.7			5 333.7	

In Table 33 the samples are given in the order taken, those marked *B.* coming from the sampling cock at the bottom of the calibrator and those marked *T.* from the top. Thus, it is seen at once that the first three samples of tail-water, taken while the mixture was being stirred, indicate that a perfect mixture had not then been secured, the heavy salt solution naturally settling toward the bottom. The samples were taken from 3 to 5 min. apart, and each has been titrated twice in the laboratory, as may be detected by examining the titrations in pairs, which are on the same line. Therefore the tail-water samples in parentheses have been omitted from the averages of the titrations. In the case of the special dilutions, the prefixed numerals relate to the salt solution sample with which the normal head-water was mixed. The second set of these, which was made from Sample 3, titrated less than the dilute salt solution, which is inconsistent. In fact, all these samples titrate too low, as they should run, on the average, about 0.28 c.c. higher than the salt solution titration, which latter averages 5 304, or they should average about 5 332, instead of 5 322.7 as they do. This discrepancy has already been discussed in determining a more probable value of R'. For these reasons, the titrations 5 260 and 5 263 have been excluded from the average, and the last set, though throwing too much weight in favor of No. 4 sample, is allowed to be included, as it is clear that the titrations of special dilutions are all too low.

Comparing now the results of the titrations in Tables 32 and 33, it will be easy to see that the former is far the more trustworthy, and for this reason a second computation of the volume of the salt-solution tank in the tests of October and November has been made, using the calibration of the calibrator of October 14th only. These latter

calculations are undoubtedly the more correct, especially that for November 23d, because there is no question as to the quantity of salt solution introduced into the salt-solution tank in that test.

The detailed calculations appear in Tables 34 (Plate XXXVI), and 35. In compiling Table 34 the volumes of samples of tail-water and special dilutions have been supposed to vary with changes in temperature in the same ratio as the specific volume of distilled water. Although this is sufficiently accurate in all cases, yet, in computing the turbine tests, it was decided to take the actual densities of tail-water and special dilutions as closely as they could be found, by using Table 1 and Plates XXXIII, XXXIV, and XXXV. The tabular densities of the salt solution have been utilized in the same way, and, as the ratios, only, of these densities are used in the calculations, no serious errors result, even where the true densities differ appreciably from the tabular values, due to the fact that commercial salt was used instead of pure NaCl.

We are now in a position to compare the best results of the chemical calibrations of the salt solution tank with the results of the calibrations based on the gravimetric determination of the volume of the calibrator in Section 64.

Plate XXXVII shows the graduation marks on the gauge-board at the side of the solution tank. Each calibration division represents a calibrator-full of water, as it was drawn from the tank, the graduation marks having been placed on the scale by using a lead pencil and carpenter's square at the level of the meniscus, which was plainly visible in the 1-in. glass tube in the slot along the center of the board.

There were three calibrations on different dates, as follows:

1.—Long marks, July 16th, 17th, 18th, 20th, and 21st, by Messrs. Ward and Bancroft;

2.—Short marks, July 30th, 31st, and August 1st, by Messrs. Ward and Bancroft;

3.—Intermediate marks, October 20th, by Messrs. Ward and Campbell.

In the chemical calibration of October, the upper level of the water in the tank was at graduation mark 12 of the short marks before the salt solution (1 calibrator-full, less about 1 800 c.c. for samples) was introduced, thus filling the tank nearly to graduation 11. At the lower level the water surface was at graduation 70 of the short marks.

TABLE 34.—Data and Preliminary Computations for the Chemical Calibrations of Iron Calibrator and Salt Solution Tank in the Tests of October 14th, 15th, 16th, and November 23d, 24th, and 25th, 1914.

No. of line.		Formula.	Symbol.	Iron Calibrator.		Salt-Solu	
						Upper level.	
				October 14th.	November 25th.	October 15th.	November 23d.
1	Temperatures: Salt solution when measured for tank			17.0	16.4	10.0	1.0
2	" " " pipetted for dilution with distilled water			17.0	19.4	10.0	11.88
3	" " " " " special dilutions			17.0	19.4	10.0	11.88
4	Distilled water for dilution of salt solution			18.75	20.0	13.0	14.75
5	Head-water for special dilutions			15.0	16.1	14.0	11.15
6	Pipetting mixture in laboratory			14.0	14.5	14.5	12.0
7	Mixture in tank			13.9	4.0	14.3	5.0
8	Head-water for diluting mixture in laboratory					14.75	11.15
9	" " in tank before introducing salt			13.9	4.0	14.0	5.0
10	Volumes: Salt solution discharged by pipette for dilute salt solution at temperature of Line 2			No. 18 791	Nos. 9 206 and 9 171	No. 18 791	Nos. 9 206 and 9 171
				9.976	9.961	9.989	9.974
11	Flask measuring distilled water at temperature of Line 4			Nos. 1 456 and 1 476	Nos. 1 407 and 1 457	Nos. 1 474 and 1 456	Nos. 1 407 and 1 457
				1 003.14	1 001.3	1 003.2	1 001.3
12	Diluted salt solution discharged by pipette at temperature of Line 4			No. 21 047	Nos. 9 206 and 9 171	No. 21 047	Nos. 9 206 and 9 171
				10.011	10.006	10.011	10.000
13	Titration per pipette of diluted salt solution sample in laboratory			54.712	53.041	55.004	53.747
14	Titration of diluted salt solution per liter in laboratory	Line 13 ÷ Line 12		5 465.2	5 300.9	5 554.8	5 374.7
15	Ratio of dilution of diluted salt solution with distilled water	Line 11 ÷ Line 10		100.56	100.32	100.03	100.03
16	Titration per liter of salt solution at temperature of Line 2	Line 14 × Line 15		549 561	531 786	558 269	539 474
17	Concentration (approximate) in grammes per liter of salt solution at temperature of Line 2			295.43	283.89	299.45	284.74
18	Percentage of salt in solution			24.86	23.73	25.2	23.60
19	Densities: Salt solution when measured at temperature of Line 1			1.1881	1.1710	1.1940	1.1799
20	" " " pipetted for dilution at temperature of Line 2			1.1881	1.1698	1.1940	1.1731
21	" " " " " special dilutions at temperature of Line 3			1.1881	1.1693	1.1940	1.1721
22	" " at temperature of mixture in tank, temperature of Line 7			1.1897	1.1764	1.1922	1.1751
23	Titration of salt solution per liter when measured for tank, temperature of Line 1	Line 16 × Line 19 ÷ Line 20		549 561	532 584	558 269	531 393
24	" " " " " at temperature of mixture in tank, temperature of Line 7	Line 16 × Line 22 ÷ Line 20	v	550 265	535 090	558 091	530 851
25	Salt solution when measured for tank at temperature of Line 1			Nos. 1 474, 1 456, 11—100 c.c.	Nos. 1 457, 1 407, 3—100 c.c.		
26	" " for tank at temperature of mixture in tank, temperature of Line 7	Line 25 × Line 19 ÷ Line 23	q	2.4069	2.4038	239.530	240.770
27	" " discharged by pipette for special dilutions, temperature of Line 3			2.4068	2.3918	239.893	241.131
28	Distilled water at temperature of mixture in tank, temperature of Line 7			No. 18 791	Nos. 9 206 and 9 171	No. 18 791	Nos. 9 206 and 9 171
29	Volumes: " " " " " special dilutions " " " " 5			9.976	9.961	9.989	9.974
30	" " " " " pipetting mixture in laboratory, temperature of Line 6			1.0007	1.0000	1.0009	1.0000
31	Flask for special dilutions at temperature of Line 5			1.0009	1.0010	1.0007	1.0004
32	Special dilutions at temperature of mixture in tank, Line 7	Line 31 × Line 26 ÷ Line 29	Q'	1.0007	1.0008	1.0008	1.0005
33	Salt solution for special dilutions at temperature of mixture in tank, Line 7	Line 27 × Line 21 ÷ Line 22	q'	Nos. 1 456 and 1 476	Nos. 1 407 and 1 457	Nos. 1 474 and 1 456	Nos. 1 407 and 1 457
34	Ratio of dilution for special dilutions	Line 32 ÷ Line 33	k'	1 003.05	1 001.2	1 003.1	1 001.1
35	Value of k' or k, approximate		k'	1 002.8	1 000.3	1 003.2	1 000.7
36	Value of k or k' "		k	9.963	9.921	9.984	9.948
				100.65	100.82*	100.49	100.59
				0.9744	0.9746	0.9727	0.9735
				0.9744	0.9746	0.9727	0.9735
37	Volume of pipette for special dilutions, temperature of Line 5			No. 21 047	Nos. 9 206 and 9 171	No. 21 047	Nos. 9 206 and 9 171
38	Titration per pipette of special dilutions			10.006	10.002	10.005	9.997
39	Titration per liter of special dilutions at temperature of Line 5	Line 38 ÷ Line 37		54.931	53.227	57.606	54.508
40	Titration of special dilutions at temperature of mixture, Line 7	Line 39 × Line 29 ÷ Line 28	v'_3	5 489.9	5 321.6	5 757.7	5 451.8
				5 490.9	5 326.9	5 757.1	5 454.0
41	Volume of pipette measuring mixture from tank, temperature of Line 6			No. 21 047	Nos. 9 206 and 9 171	No. 21 047	Nos. 9 206 and 9 171
42	Titration per pipette of Line 41			10.005	10.001	10.005	9.998
43	Titration per liter of mixture at temperature of Line 6	Line 42 ÷ Line 41		55.004	53.227	56.808	55.215
44	" " " " " " " " mixture, Line 7	Line 43 × Line 30 ÷ Line 28	v_3	5 497.7	5 322.2	5 678.0	5 522.6
				5 497.7	5 327.5	5 678.0	5 525.4

* This ratio not being in reasonable agreement with the concentrations of salt solution and special dilutions, is probably in error. Based upon the concentrations, the ratio should be 100.97, which therefore has been adopted for this

INARY COMPUTATIONS FOR THE CHEMICAL CALIBRATIONS OF IRON CALIBRATOR AND SALT S
TESTS OF OCTOBER 14TH, 15TH, 16TH, AND NOVEMBER 23D, 24TH, AND 25TH, 1914.

	Formula.	Symbol.	Iron Calibrator.		
			October 14th.	November 25th.	O
.....	.....	....	17.0	18.4	
.....	.....	....	17.0	19.4	
.....	.....	....	17.0	19.4	
.....	.....	....	18.75	20.0	
.....	.....	....	15.0	16.1	
.....	.....	....	14.0	14.5	
.....	.....	....	13.9	4.0	
.....	.....	....	13.9	4.0	
e of Line 2.....	.....	....	No. 18 791 2.976	Nos. 9 206 and 9 171 2.981	
.....	.....	....	Nos. 1 456 and 1 476 1 008.14	Nos. 1 407 and 1 457 1 001.3	Nos.
.....	.....	....	No. 21 047 10.011	Nos. 9 206 and 9 171 10.006	
.....	.....	....	54.712	53.041	
.....	Line 13 ÷ Line 12	....	5 465.2	5 300.9	
.....	Line 11 ÷ Line 10	....	100.56	100.32	
.....	Line 14 × Line 15	....	549 581	531 786	5
2.....	.....	....	295.43	265.89	
.....	.....	....	24.86	22.72	
.....	.....	....	1.1881	1.1710	
.....	.....	....	1.1881	1.1693	
3.....	.....	....	1.1881	1.1693	
.....	.....	....	1.1897	1.1764	
.....	Line 16 × Line 19 ÷ Line 20	....	549 581	532 584	5
e 7.....	Line 16 × Line 22 ÷ Line 20	v	550 295	535 080	5
.....	.....	....	Nos. 1 474, 1 456, 11—100 c. c.	Nos. 1 457, 1 407, 3—100 c. c.	
.....	.....	....	2.4069	2.4028	
f Line 7.....	Line 25 × Line 19 ÷ Line 22	q	2.4088	2.3918	
Line 3.....	.....	....	No. 18 791	Nos. 9 206 and 9 171	
7.....	.....	....	2.976	2.981	
5.....	.....	....	1.0007	1.0000	
erature of Line 6..	.....	....	1.0009	1.0016	
.....	.....	....	1.0007	1.0008	
.....	.....	....	Nos. 1 456 and 1 476	Nos. 1 407 and 1 457	Nos.
e 7.....	Line 31 × Line 28 ÷ Line 29	Q'	1 008.05	1 001.2	
e 7.....	Line 27 × Line 21 ÷ Line 22	q'	1 002.8	1 000.2	
.....	Line 32 ÷ Line 33	R'	9.963	9.921	
.....	.....	k'	100.65	100.82*	
.....	.....	k	0.9744	0.9746	
.....	.....	....	0.9744	0.9746	
.....	.....	....	No. 21 047	Nos. 9 206 and 9 171	
.....	.....	....	10.005	10.002	
.....	.....	....	54.931	53.227	
.....	Line 38 ÷ Line 37	....	5 489.8	5 321.6	
.....	Line 39 × Line 29 ÷ Line 28	v'_2	5 490.9	5 326.9	
.....	.....	....	No. 21 047	Nos. 9 206 and 9 171	
.....	.....	....	10.005	10.001	
.....	.....	....	55.004	53.337	
.....	Line 42 ÷ Line 41	....	5 497.7	5 333.2	
.....	Line 43 × Line 30 ÷ Line 28	v_2	5 497.7	5 337.5	

ution and special dilutions, is probably in error. Based upon the concentrations, the ratio should be 100.97, which

ALT SOLUTION TANK IN THE

SALT-SOLUTION TANK.			
Upper level.		Lower level.	
October 15th.	November 23d.	October 16th.	November 24th.
10.0	1.0	13.9	— 2.0
10.0	11.68	14.0	18.4
10.0	11.88	14.0	18.3
19.0	14.75	17.25	18.0
14.0	11.15	15.0	14.84
14.5	12.0	15.3	14.0
14.2	5.0	15.9	7.5
14.75	11.15	.....	.....
14.0	5.0	15.3	8.5
No. 18 791	Nos. 9 206 and 9 171	No. 18 791	Nos. 9 206 and 9 171
9.969	9.974	9.974	9.980
Nos. 1 474 and 1 456	Nos. 1 407 and 1 457	Nos. 1 474 and 1 456	Nos. 1 457 and 1 407
1 003.2	1 001.2	1 003.2	1 001.3
No. 21 047	Nos. 9 206 and 9 171	No. 21 047	Nos. 9 206 and 9 171
10.011	10.000	10.009	10.004
55.604	52.747	52.175	52.999
5 554.3	5 274.7	5 212.8	5 297.8
100.63	100.38	100.55	100.33
558 929	529 474	524 303	531 528
300.45	264.74	281.84	265.76
25.2	22.60	23.55	22.69
1.1940	1.1768	1.1813	1.1791
1.1940	1.1721	1.1812	1.1699
1.1940	1.1721	1.1812	1.1699
1.1923	1.1751	1.1804	1.1749
558 929	531 593	524 355	535 727
558 091	530 851	523 936	533 814
		No. 1 476	Nos. 1 457 and 1 407
239.530	240.770	20.067	21.516
239.889	241.181	20.068	21.594
No. 18 791	Nos. 9 206 and 9 171	No. 18 791	Nos. 9 206 and 9 171
9.969	9.974	9.974	9.980
1.0008	1.0000	1.0010	1.0001
1.0007	1.0004	1.0009	1.0003
1.0008	1.0005	1.0009	1.0007
Nos. 1 474 and 1 456	Nos. 1 407 and 1 457	Nos. 1 474 and 1 456	Nos. 1 457 and 1 407
1 003.1	1 001.1	1 003.1	1 001.2
1 003.2	1 000.7	1 003.2	1 000.5
9.964	9.949	9.961	9.937
100.48	100.59	100.51	100.64
0.9737	0.9755	0.9737	0.9763
0.9737	0.9755	0.9737	0.9763
No. 21 047	Nos. 9 206 and 9 171	No. 21 047	Nos. 9 206 and 9 171
10.005	9.997	10.006	10.001
57.606	54.502	55.174	55.960
5 757.7	5 451.8	5 514.1	5 595.4
5 757.1	5 454.0	5 513.6	5 592.5
No. 21 047	Nos. 9 206 and 9 171	No. 21 047	Nos. 9 206 and 9 171
10.005	9.996	10.006	10.000
56.808	53.215	51.607	59.591
5 678.0	5 523.6	5 157.6	5 959.1
5 678.0	5 525.4	5 157.1	5 962.7

, which therefore has been adopted for this calibration.

TABLE 23.—(Continued).

TABLE 35.—FINAL COMPUTATIONS FOR THE CHEMICAL CALIBRATIONS OF IRON CALIBRATOR AND SALT-SOLUTION TANK IN THE TESTS OF OCTOBER 14TH, 15TH, 16TH, AND NOVEMBER 23D, 24TH, 25TH, 1914.

Line.		Iron Calibrator.		Salt-Solution Tank.			
				Upper level.		Lower level.	
		Oct. 14.	Nov. 25.	Oct. 15.	Nov. 23.	Oct. 16.	Nov. 24.
1	k''	0.976	0.975	0.972	0.978	0.976	0.979
2	c_2 ...	5 498.00	5 337.50	5 678.0	5 525.4	5 157.1	5 962.7
3	(See Equations 106) c'_2	5 490.90	5 326.90	5 757.1	5 454.0	5 513.6	5 899.5
4	$k'' c_2$	5 366.048	5 204.06	5 519.0	5 403.8	5 033.3	5 837.5
5	c	550 295.0	535 030.00	558 091.	530 851.	523 936.	533 814.
6	$c - k'' c_2$	544 928.95	529 825.9375	552 600.	525 000.	518 900.	527 976.
7	$c_2 - c'_2$	7.100	10.600	−79.1	71.4	−356.5	63.2
8	$f = \left(c_2 - c'_2\right) \div \left(c - k'' c_2\right)$....	0.000013029	0.0000200	−0.0001431	0.000136	−0.0006870	0.000120
9	R'	100.65	100.97	100.48	100.59	100.51	100.64
10	$R'f$	0.00131	0.00202	−0.01438	0.0137	−0.06905	0.0121
11	$r' = R' - k'$	99.68	100.00	99.508	99.612	99.534	99.661
12	$r = r' \div (1 + R'f)$	99.55	99.80	100.96	98.266	106.92	98.470
13	k	0.974	0.97	.....	.....	0.976	0.979
14	$R = r + k$	100.524	100.77	.....	.....	107.90	99.449
15	q	2.4088	2.3918	.....	.....	20.083	21.594
16	$Rq = Q$ at temperature of mixture in tank.	241.64 @ 13.9° cent.	241.02 @ 4° cent.	.....	.....	2 167.0	2 147.5
	Correction	.....	−0.04	.....	.....	.....	.....
17	Corrected	.....	240.98 "	.....	.....	.....	.....
18	Volume of calibrator at 13.9° cent.	241.64	241.06	.....	.....	.....	.....

TABLE 35.—(*Continued*).

Line.		Salt-Solution Tank.			
		Upper level.		Lower level.	
		Oct. 15.	Nov. 23.	Oct. 16.	Nov. 24.
	Calculation of r'', r_x, and R_x.				
19	Temperature of head-water for diluting mixture in laboratory	14.75	11.15		
20	" " mixture from tank for dilution	14.	13.		
21	" " mixture in tank	14.2	5.		
22	Volume of ½-liter flask at temperature of Line 19	500.53	500.24		
23	" " 1 " " " " " " " 20	1 003.1	1 001.1		
24	Density of normal head-water at temperature of Line 19	999.4	999.4		
25	" " " " " " " " " " 21	999.5	1 000.0		
26	" " mixture at temperature of Line 20	1 001.3	1 002.8		
27	" " " " " " " " 21	1 001.3	1 003.2		
28	Vol. of N. H. water for special dilutions at temperature L. 21 = L. 22 × L. 24 ÷ L. 25	500.48	499.94		
29	Vol. of mixture for special dilutions at temperature L. 21 = L. 23 × L. 26 ÷ L. 27	1 003.1	1 000.7		
30	r'' L. 28 ÷ L. 29	0.4989	0.4996		
31	k_x	0.974	0.973		
32	$r_x = (r - r'' k_x) \div (1 + r'')$ See Equations (83)	67.032	.....		
33	$R_x = (r + k_x) \div (1 + r'')$	.....	66.177		
34	q = mean of two iron calibrator volumes, Line 18 at 13.9° C.	241.35	241.35		
35	q reduced to temperature of salt solution when measured for tank	239.53 at 10° cent.	240.77 at 1° cent.		
36	q " " " " mixture in tank. (See Lines 25 and 26, Table 34.)	239.89 at 14.2° cent.	241.13 at 5° cent.		
37	$Q = r_x q$ and $R_x q$, respectively, based on vol. iron calibrator, October 14 and November 25	16 081.	15 957.		
38	q = vol. of calibrator as found on October 14. (See Line 18)	241.64 at 13.9° cent.	241.64 at 13.9° cent.		
39	q after deducting 1.8 and 0.48 for samples, respectively, and reducing to temperature of salt solution when measured for tank	239.82 at 10° cent.	241.06 at 1° cent.		
40	q reduced to temperature of mixture in tank, Line 39 × Line 19 / Line 22 (Table 34)	240.18 at 14.2° cent.	241.40 at 5° cent.		
41	$Q = r_x q$ and $R_x q$, respectively, based on vol. iron calibrator, October 14 only.	16 100.	15 975.		

In the chemical calibration of November, at the upper level, the water surface was at the point marked *A*, Plate XXXVII, slightly below graduation 13 of the long marks; and, at the lower level, the water surface was at graduation 70 of the long marks.

Therefore, to compare the two chemical calibrations, it is necessary to reduce them to the same levels, which, for definiteness, will be at graduations 12 and 70 of the long marks. These corrections, therefore, are due to discrepancies of 0.17 in. (scaled from Plate XXXVII) between the long and short marks for graduation 70, and 0.03 in. between the long and short marks for graduation 12, the average lengths of division for 1 calibrator-full at these levels being, respectively, 1.41 and 1.49 in., giving percentage corrections of $17 \div 1.41 = 12\%$ and $3 \div 1.49 = 2\%$, correspondingly. The capacity of a calibrator being approximately 240 liters, the corrections in liters are:

For upper level = 2% of 240 = 5 liters;
For lower level = 12% of 240 = 29 liters.

In both cases these corrections should be deducted from the volumes given by the chemical calibration of October, to reduce to the proper graduation mark.

In the case of the chemical calibration of November, a correction must be added to the volume at the upper level to obtain the corresponding volume at the long mark for division 12. The correction to reduce from the mark, *B*, to long graduation mark 12 is due to a discrepancy of level of 0.3 in. in an average length of 1.50 in., or exactly 20% of 1 calibrator-full, which amounts to 48 liters to be added to the November volume at the upper level.

Table 36 gives the adjustment of the figures from Table 35.

It must be remembered that the November calibration here is the more truthworthy, being 13 875 liters between graduation marks 12 and 70.

To obtain figures for comparison from the gravimetric calibrations of the calibrator and the calibration scales of Plate XXXVII, an account must be made of the temperatures at which the scales on the drawing were made. The calibration at long marks in July was made at an average temperature of 20½° cent.; and those in October, at the intermediate marks, were at 13½° cent. The short marks will be neglected at present for the reason that there is some doubt about them, as will

appear in the next section. The capacity of the calibrator in this case is about the same as in the case of the long marks, the temperature being 19¾° cent. Thus

Volume of calibrator by Fig. 5 = 239.72 liters at 20½° cent.
= 239.66 liters at 13¼° cent.,

from which

(July 18th) 58 calibrator-fulls at 20½° = 13 904 liters.
(October 20th) 58 calibrator-fulls at 13¼° = 13 900 liters.

TABLE 36.—ADJUSTMENT OF VOLUMES DETERMINED CHEMICALLY FOR COMPARISON WITH VOLUMES DETERMINED WITH THE CALIBRATOR AND ITS OWN GRAVIMETRIC CALIBRATION.

	October 15th.	November 23d.
Volume at upper level. Table 35 (based on volume of calibrator, October 14th only)	16 100*	15 975
Corrections from above	— 05	+48
Resulting volume for long mark 12	16 095	16 023
Volume at lower level. Table 35	2 167	2 148
Correction from above	29	.....
Resulting volume for long mark 70	2 138†	2 148†
Net volume between long marks 12 and 70, 58 calibrator-fulls	13 957	13 875

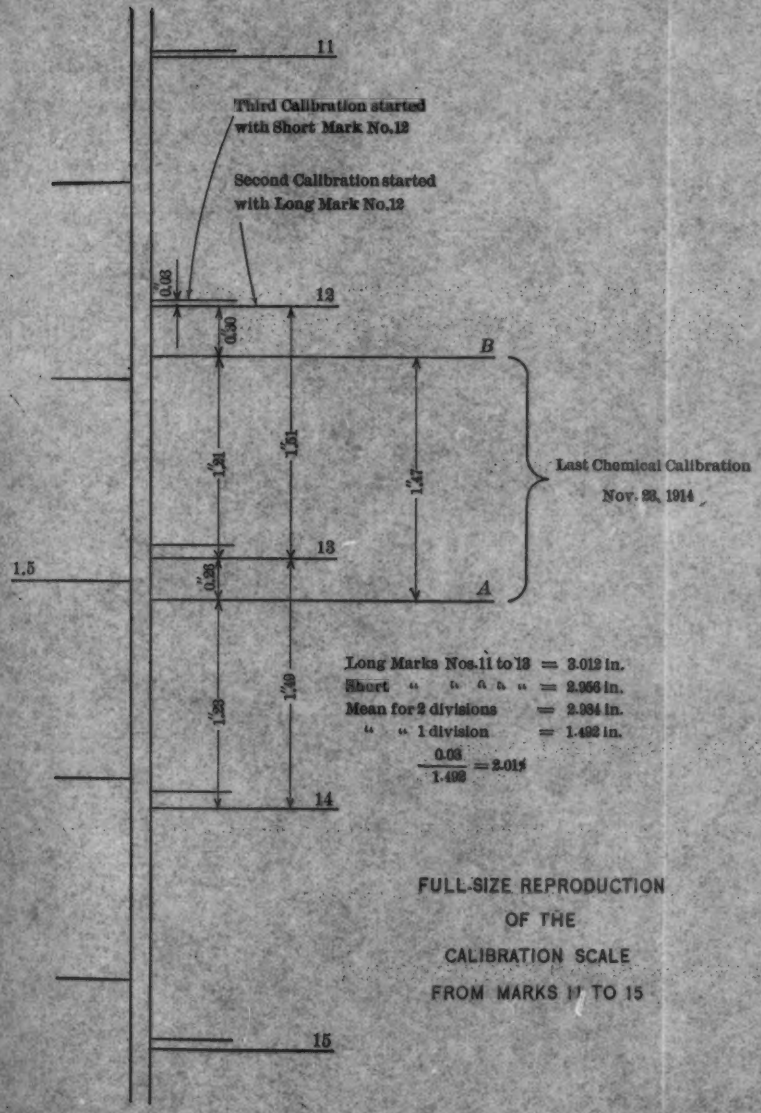
* The reader is reminded that there was no exact measurement of the quantity of salt solution taken for samples from the contents of the iron calibrator while it was being emptied into the salt-solution tank on October 15th.

† An error of 0.004 or 0.005 ft. in reading the tank gauge would introduce an error of 8 or 10 liters in the volume computation.

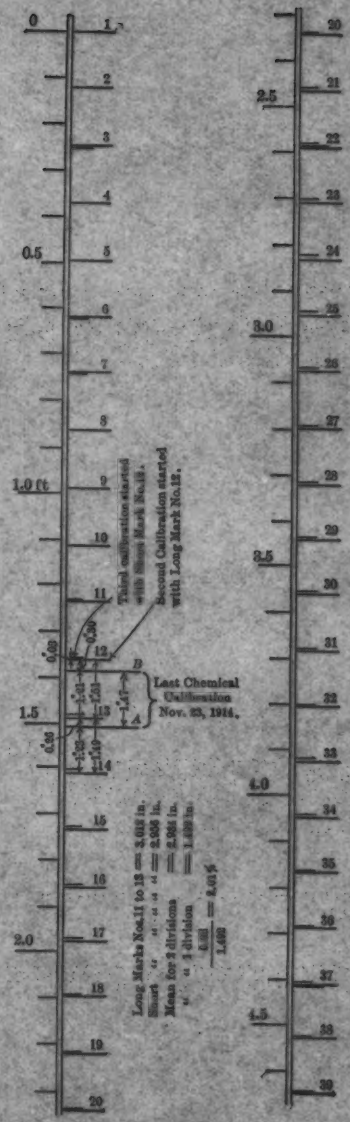
Now, the third calibration (intermediate marks) started with short mark 12 and ended about 0.05 in., or 5 ÷ 1.41 = 3½%, of 1 calibrator-full, 8¼ liters, above graduation mark 70. That is to say, the third calibration indicates that the volume from long mark 12 to long mark 70 at the time of the third calibration was 13 900 + 8¼ — 5 = 13 903¼ liters, or the two calibrations (first and third) check within ¼ liter, 58 calibrator-fulls in July, therefore, being 13 904 liters, which is what the volume of the tank from long mark 12 to long mark 70 was in July by the calibration at that time. Hence, the following comparison results:

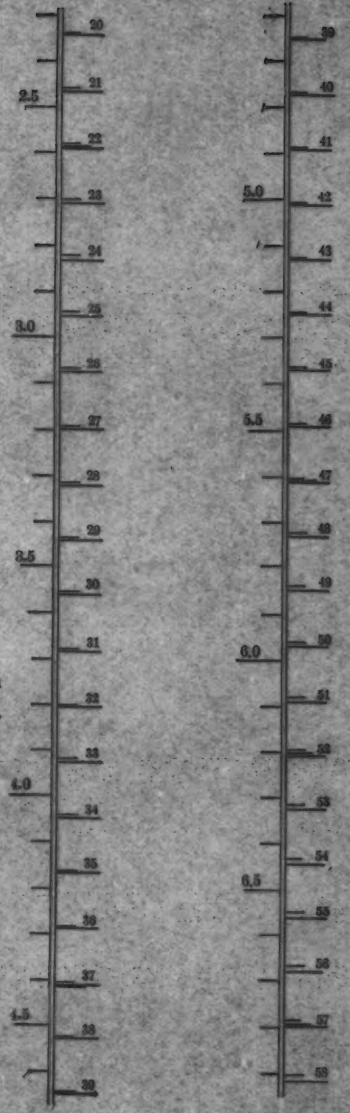
Volume of tank between graduation marks 12 and 70, of the first calibration by the calibrator, based on both the first and third calibrations..... 13 904 liters.

Volume of tank between these same marks..... (October).... 13 957 "
but by the chemical method..... (November). 13 875 "
Mean of last two..... 13 916 "



FULL-SIZE REPRODUCTION
OF THE
CALIBRATION SCALE
FROM MARKS 11 TO 15





CALIBRATION SCALES
AT SIDE OF
SALT SOLUTION TANK

Long Calibration Marks, No.1, used on Tests *G, H, I, K, L, N,* and *O* on Unit No.12.
Short Calibration Marks, No.2, used on all other tests.

Calibration No.1. Long Marks.
Made July 16th, 17th, 20th, 21st, and 22nd, 1914.
Average Temperature = 22.15° Cent.

Calibration No.2. Short Marks
Made July 28th, 31st, and August 1st, 1914.
Average Temperature = 19.64° Cent.

Calibration No.3 Marks ¾ in. long.
Made October 19th, 1914.
Average Temperature = 13.25° Cent.

Long Marks Nos. 69 to 71 = 2.956 in.
Short " " " " = 2.839 "
Mean for 2 divisions = 2.898 "
" " 1 division = 1.414 "
0.17 / 1.414 = 12.02%

Apparently, therefore, there was no change in volume of the 58 divisions of the tank by the first and third gravimetric calibrations, but there is considerable change, 82 liters in 13 900, or 0.6%, by the two chemical calibrations, the tank being smaller in November than in October. Attention has been called to the fact that the chemical calibration at the upper level in October is not so trustworthy as the later one, on account of the failure to measure exactly the quantity of salt solution taken from the calibrator for samples. If we neglect this, we may say that the chemical determination indicates a volume of 13 875 liters in November, as against 13 904 liters in July, by means of the calibrator, or only 0.2% discrepancy, and this is in the direction which one might suppose such a change would be.

67.—*Degree of Precision in Turbine Tests.*—Attention has been called to the fact that the chemical calibrations were not conducted under such favorable conditions as the best of the turbine tests. Accordingly, these discrepancies of 0.2 to 0.6% are larger than the discrepancies of the best turbine tests by a considerable margin. However, there is one important conclusion to be drawn from these chemical calibrations, namely: There was no serious systematic error involved in the chemistry and laboratory operations during the turbine tests. In so far as the chemical procedure was concerned, the results of the turbine tests are correct to a small fraction of 1 per cent.

It may also be added that the mean of the two chemical tests checks the mean of the first and third calibrations by the calibrator within 0.1 of 1 per cent. If the second calibration with the calibrator is included, the error is still less, but no importance can be attached to these facts. The calculation, however, follows:

Fifty-eight calibrator-fulls in the second calibration (short marks) at 19¼° cent. are equivalent to $58 \times 239.716 = 13\,903\frac{1}{2}$ liters. To reduce this to long marks 12 and 70, simply add, algebraically, the corrections due to the discrepancies in elevation of the corresponding short marks, which have already been ascertained, namely, 29 and 5 liters, respectively. Thus the volume between the long marks 12 and 70, based on the second calibration with the calibrator is $13\,903\frac{1}{2} + 29 - 5 = 13\,928$ liters.

The mean of the values of the volume between divisions 12 and 70, based on all three calibrations with the calibrator, therefore, is the mean of 13 904, 13 904, and 13 928, which is 13 912 liters, as com-

pared with a mean of 13 916 liters by the two chemical calibrations. This omits entirely the last chemical calibration of the contents of the calibrator because of the inconsistencies in the chemical treatment previously mentioned.

We might make other comparisons by various combinations of results, but these would not improve matters, even if they gave closer agreement between the results by the two methods. The writer prefers to adopt the values which can be least criticised. These are based on the calibrator volume determined on October 14th, the tank calibration at the upper level, on November 23d, and the mean of the two calibrations at the lower level, on October 16th and November 24th, both of which appear to be trustworthy. Neglecting the effects of differences of temperature, this estimate would be:

Volume at upper level, November 23d.....16 023 liters
Mean at lower level October 15th, November 23d.. 2 143 "

Probable volume, 58 divisions.....13 880 liters.

This agrees well with the gravimetric calibrations of either July or October, the mean of which is 13 902.

To make matters still clearer, Table 37 gives a compilation of the titrations of the samples taken during the chemical calibrations of the large tank.

TABLE 37.—COMPARISON OF TITRATIONS FOR CALIBRATIONS OF LARGE TANK AT THE UPPER LEVEL ON OCTOBER 15TH AND NOVEMBER 23D.

(All Units in Hundredths of a Cubic Centimeter.)

	Dilute Salt Solutions.				Special Dilutions.				Tail-Water Samples.			
	October 15th.		November 23d.		October 15th.		November 23d.		October 15th.		November 23d.	
	5 555	5 550	5 276	5 265	5 755	5 755	5 433	5 430	5 680	5 675	5 529	5 520
	5 508	5 560	5 250	5 250	5 760	5 765	5 479	5 480	5 685	5 682	5 525	5 527
	5 560	5 565	5 299	5 304	5 770	5 770	5 430	5 433	5 675	5 680	5 518	5 515
	5 565	.....	5 251	5 250	5 755	5 750	5 440	5 430	5 680	5 685	5 535	5 535
	.....	.....	5 279	5 282	5 765	.....	5 471	5 457	5 685	.....	5 517	5 512
	.....	.....	5 300	5 290	.....	.....	5 465	5 455	.....	.....	5 510	5 516
Means....	5 562	5 558.3	5 275.8	5 273.5	5 761.2	5 760	5 453	5 447.5	5 681	5 680.5	5 522.3	5 520.7
Average..	5 560.4		5 274.7		5 760.6		5 450.2		5 680.8		5 521.5	

TABLE 38.—Comparison of Titrations for Calibrations of Large Tank at the Lower Level on October 16th and November 24th.
(All Units in Hundredths of a Cubic Centimeter.)

	Dilute Salt Solutions.				Special Dilutions.				Tail-Water Samples.			
	October 16th.		November 24th.		October 16th.		November 24th.		October 16th.		November 24th.	
	5 210	5 215	5 302	5 298	5 520	5 518	5 897	5 895	5 157	5 165	5 927	5 930
	5 220	5 220	5 300	5 301	5 515	5 515	5 905	5 900	5 160	5 162	6 004	5 973
	5 220	5 220	5 300	5 303	5 520	5 523	5 901	5 900	5 160	5 160	5 945	5 945
	.....	.....	5 300	5 295	5 520	5 511	5 903	5 900	.....	.....	5 945	5 955
	.....	.....	.....	.....	5 515	.....	5 905	5 910	.....	.....	5 965	5 960
	.....	.....	.....	.....	.....	.....	5 880	5 856	.....	.....	5 970	5 990
Means...	5 216.7	5 218.3	5 300.5	5 299.3	5 518	5 516.8	5 898.5	5 893.5	5 159	5 162.3	5 959.3	5 958.8
Average.	5 217.5		5 299.9		5 517.4		5 896.0		5 160.7		5 959.1	

It is apparent from Tables 37 and 38 that the chemical work of November was not as good as that of October, and, perhaps, one of the most potent reasons therefor is the fact that most of the outdoor work in November was done in bad weather, with the thermometer near zero, Fahrenheit. On the other hand, there is a larger number of titrations in the November work, which would tend to reduce the residual accidental error in the laboratory observations.

As a general rule, two or three chemists performed the titrations in pairs from the same sample, mates appearing on the same horizontal line of the tables.

In closing this section it may be remarked that there is no basis for comparing the gravimetric with the chemical calibration of the calibrator. There is considerable difference between the two, but this is as it should be, as the gravimetric calibration was a measure of the delivery and the chemical calibration was a measure of the total contents of the calibrator. The latter should exceed the former, as it does, but there were no weighings of the total contents of the calibrator, so that there is really no basis for comparison. The evidence is that the two methods can be made to check each other within a few hundredths of 1%, even when the operations are not nearly so refined as the more painstaking work of a laboratory research.

68.—Changes in the Capacity of the Tank.—The general method of calibrating the solution tank was explained in Section 62 *et seq.*

Plate XXXVII is a *fac simile* of the calibration scale taken directly from the gauge-board after it had been removed from the tank at the close of the tests.

The first calibration was made on July 16th, 17th, 18th, 20th, and 21st, 1914. The corresponding scale is indicated on Plate XXXVII by the longest of the graduation marks. The second calibration, July 30th, 31st, and August 1st, 1914, is indicated by the shortest marks; and the third calibration is indicated by the marks of intermediate length.

The first and second calibrations covered the entire scale from graduation 1 to graduation 76, but the third calibration began at graduation 12 and continued to graduation 70, a graduation mark being made at every fifth division.

It will be observed that the first and third calibrations agree remarkably well, and that the second runs with increasing divergence on the side of larger capacity when compared with either of the other two. There is, however, a close agreement between the first and second calibrations from graduation 1 to graduation 45 or 46, and from 52 or 54 to the end of the scale.

Thus all the calibrations agree well with one another except that the first and second calibrations do not agree at points between 45 and 54. Therefore, it must be concluded that there was either an actual change in the calibration scale, from 45 to 54 between the first and second calibrations, followed by a return to original conditions, or an error was made along this part of the scale during the second calibration.

It is not to be imagined that the temperature changes could affect the calibrations so as to cause any such differences. The tank was of 2-in. white pine, with steel hoops. It was about 10 ft. high and about 9 ft. 4 in. in diameter at the top and 9 ft. 8 in. diameter at the bottom, inside measure. The temperatures and corresponding average volumes of the calibrator in the first and second calibrations are given in Table 39. This shows that the changes in the capacity of the calibrator are limited to 0.01 or 0.02 of 1%, which is considerably within the limit of error in the observations during the turbine tests. Indeed, the indications are that the volume of the tank was much more consistently constant than could have been anticipated. The third calibration, considered in conjunction with the first, shows that the volume of the tank was the same on both occasions, and the second calibration points to either a change or a slight error between graduations 45 and 54.

TABLE 39.—Average Volume of Calibrator.

Date.	Graduation.	Temperature, in degrees, centigrade.	Average volume of calibrator, in liters.
First Calibration:			
July 16th	1-14	21.0	239.73
17th	14-32	21.4	
18th	32-45	21.0	
20th	45-60	19.0	239.71
21st	60-76	19.0	
Second Calibration:			
July 30th	1-10	19.5	239.715
31st	10-44	19.8	
August 1st	44-76	19.6	
Third Calibration:			
October 20th	12-70	13.2	239.665

The writer is inclined to the view that each calibration should be considered as standing by itself, as being the best criterion of the actual volume of the tank at the time of the calibration, any error in utilizing the calibrations being in all probability less than 0.1 of 1 per cent. Therefore, the new calibrations were used as soon as made, the older ones being disregarded in the ensuing turbine tests.

69.—Effect of Painting on the Capacity of the Tank.—Prior to the time of painting the salt-solution tank, it was found that readings, taken night and morning, showed variations of from 0.03 to 0.06 ft. Apparently, there were diurnal changes in capacity due to periodical variations of temperature, humidity, and other atmospheric conditions. Therefore, it was decided to give the tank two coats of white lead.

After the tank had been painted, these fluctuations of water level became markedly lessened, and toward the close of the tests the capacity of the tank became almost fixed.

Table 40, compiled from the record, shows characteristic observations on the level of the water in the tank.

The solution drawn off between the observations of September 25th and 26th was consumed in running some tests on the large centrifugal pump to ascertain the power required for this purpose during a regular turbine test.

TABLE 40.—OBSERVATIONS ON LEVEL OF WATER IN SALT-SOLUTION TANK.

Condition.	Date.	Time.	Gauge reading, in feet.	Remarks.
Before painting.	July 8th.	9.40 A. M.	10.33	
	" 8th.	3 P. M.	10.365	
	" 9th.	3 P. M.	10.38	
After one coat of white lead.	July 15th.	2 P. M.	10.08	
	" 15th.	5 P. M.	10.04	
	" 16th.	8 A. M.	10.02	
	July 20th.	5 P. M.	7.13	
	" 21st.	8 A. M.	7.13	Tank filled.
After second coat of white lead.	July 21st.	5 P. M.	10.11	
	" 22d.	8 A. M.	10.10	Still later, after refilling.
	July 28th.	4 P. M.	10.53	
	" 29th.	8 A. M.	10.50	
	" 29th.	1.30 P. M.	10.52	
	" 29th.	5.30 P. M.	10.53	
	" 30th.	8 A. M.	10.52	
Another series.	August 5th.	2 P. M.	10.48	
	" 5th.	5.30 P. M.	10.48	
	" 6th.	8.30 A. M.	10.47	
	" 6th.	3.15 P. M.	10.49	
	" 7th.	8 A. M.	10.49	
Last long series on same tank full of salt solution.	September 18th.	5.30 P. M.	9.95	
	" 19th.	8.30 A. M.	9.95	
	" 19th.	12.01 P. M.	9.95	
	" 21st.	8.30 A. M. and P. M.	9.95	
	" 22d	8.30 A. M. and P. M.	9.95	
	" 23d	8.30 A. M. and P. M.	9.95	
	" 24th.	8.30 A. M. and P. M.	9.95	
	" 25th.	8.30 A. M. and P. M.	9.95	
	September 26th.	8.30 A. M. and P. M.	9.40	Some salt solution drawn off.
	" 27th.	8.30 A. M. and P. M.	9.40	
	" 28th.	8.30 A. M. and P. M.	9.40	

These observations, together with the results of the calibrations, show that there were no serious variations in the capacity of the salt-solution tank during the series of tests on the turbines.

70.—Care of the Tank.—The principal rule to enforce in the care of the tank is as to the load it carries. If it is always kept filled with salt solution to about the same level during a series of tests, its volume has no tendency to change seriously. If there is no system in this matter, it will be uncertain as to whether there have been serious changes in capacity.

Accordingly, it was a rigid rule, in the tests described, to keep the tank filled with salt solution as continuously as possible. It was always the aim to fill the tank with salt solution immediately after each test,

whenever possible, and on only a few occasions was it allowed to stand over night filled with fresh water instead of salt solution. At no time was it allowed to remain empty, or partly full, during the night, or for any longer time during the day than could be avoided.

The writer attributes the constancy of volume of the salt-solution tank in large measure to the fact that it was kept continuously full of salt solution.

71.—The calibrations of the electrical instruments are not reproduced in this paper, but are filed in the Library of the Society for reference by any one interested.

72.—Error 9.—Time.—An ordinary watch will keep time sufficiently well for the purpose of turbine testing, even when time is to be determined to within 0.1, or even 0.01 of 1% in precision. There are 1 440 min. in a day, so that 1.44 min., or 1 min., 26.4 sec., represents 0.1 of 1% and 8.64 sec. represents 0.01 of 1 per cent.

Therefore, it may be stated that an ordinary watch in only fair regulation will run within 0.1 of 1%, but it must be in good regulation to run within 0.01 of 1%, and it should be compared with a reliable standard before and after each test, when it is desired to ascertain time more exactly than within 0.1 of 1% in precision.

A much more serious matter than the systematic error in the rate of the watch is the accidental and personal errors of observation. These have been discussed to some extent in Section 63, and will not require extended comment here, except to call attention to the possibilities of watches and chronographs.

There is considerable doubt about the rates of watches operating near large electrical machines, especially in case of direct current. For this reason, it may be advisable, in some cases, to use a chronograph driven electrically by a clock at a distance. A chronograph may be advisable on other grounds, especially where tests can be of short duration, thus saving labor and chemicals where the chemical method is used.

When a watch is used it must be considered that accidental errors approaching 1 sec. are not only possible but probable, even with a stop-watch. Moreover, stop-watches are likely to change their rates by 1 or 2 sec. in 15 or 20 min., unless they are very carefully handled, and there is a high probability of one failing entirely in the course of a test. It is important, therefore, where reliance is placed on stop-watches, to keep several running during a test—not less than three—

where it is important not to lose the observations altogether, and these watches should be rated and compared before and after each use.

In order to secure sufficiently accurate time observations with stop-watches, it must be considered that an accidental error of at least 1 sec. may be made in starting and stopping the watch, or the split-second hand where a split-second watch is used, unless several observers with different watches observe at both start and finish. In this case an idea of the error of observing can be gained from the separate estimates made by the different observers; but, in the case of only one watch, the error must be counted at 1 sec., thus determining the length of a test when the limit of error is decided on.

Therefore, if the limit of the degree of error is 0.1 of 1%, it will be necessary to run the test for 1 000 sec., say, 16 or 17 min. Perhaps 15 min. is the limit, in round numbers. Accordingly, the regular turbine tests described lasted from 15 min. to 1 hour.

The kind of watch most convenient for such work has two hands: one is driven continually from the time the watch is started until it is stopped; the secondary hand moves in coincidence with the primary until it is stopped at any instant by pressure on a small stop. A second pressure on the stop causes the secondary hand to spring into coincidence with the primary again, and these operations may be repeated as often as necessary. Consequently, as many time readings as desired may be made by such a watch, and this without losing account of the correct time, which would be the case by stopping the entire mechanism, as is necessary with many kinds of stop-watches.

Another advantage may be gained, where the time is required in seconds, by having the dial graduated centesimally, each division of what is usually called the minute-scale thus representing 100 sec. Not many watches of this kind have been made, but the writer succeeded in having one divided in this way, and found that it pays well, by reducing the labor of final calculations.

73.—Error 10.—Elimination of the Density of Feed-Water.—The density of the feed-water for the turbine may not be known. If the test is purely for efficiency, where the actual discharge of the turbine is required merely as a factor in the efficiency formula, the density of the feed-water need not be determined, if the units of volume of the calibrator and tank are computed from weights of actual feed-water in conjunction with densities of distilled water.

The error thus made may be eliminated in the efficiency calculation by charging the turbine with theoretical power based on the densities of distilled water and the observed discharge, as given by the observations on the calibration scale at the side of the tank, the counter-errors, thus made, neutralizing each other in the efficiency computation.

Thus, if the density of distilled water is d, that of the actual feed-water is d_1, and the weight of the contents of the calibrator when filled with feed-water is w, the true volume and the computed volume, based on the density of distilled water, are, respectively:

$$\text{True volume of calibrator} = w \div d_1.$$
$$\text{Computed } \text{“} \quad \text{“} \quad \text{“} \quad = w \div d.$$

Hence the ratio of the computed to the true volume is $d_1 \div d = 1 + e$, where $100e$ is the percentage of error relative to the true volume. That is, all volumes measured by the calibrator will be affected by the same percentage of error; or, if V is the true discharge of the turbine during a test, then the observed volume will be:

$$\text{Observed volume} = V\,(1 + e).$$

Now, the power with which the turbine will be charged, based on the density of distilled water, and the discharge determined by the calibrator, is:

$$\text{Observed power} = \frac{(V\,[1 + e])\,d\,h}{550} \quad \ldots\ldots\ldots\ldots (141)$$

where h is the head acting on the turbine;

but $$d_1 = d\,(1 + e),$$

as has just been shown, therefore,

$$\left.\begin{matrix}\text{True power charge-} \\ \text{able to turbine...}\end{matrix}\right\} = \frac{V\,d_1\,h}{550} = \frac{V\,(1 + e)\,d\,h}{550} \quad \ldots. (142)$$

or, this method of calibrating the tank, observing the tests and computing the results, gives the true efficiency, though there may be a small error in the volume of the tank, and therefore of the discharge, as shown by the calibration scale on the gauge-board.

74.—Error 11.—Omission of Weight of Salt.—When the method of weights, Section 10, is used, there is no error due to the omission of the weight of salt, as the total weight of solution passing through the turbine is determined by the equations based on weights (Equations (11) to (28), inclusive). There may be an insignificant error, however, due to

the fact that the tables are based on chemically pure NaCl, whereas the salt used is of commercial grade only. This error is entirely negligible.

In the case of the volumetric method, however, beginning with Equations (38), where the methods of Section 73 are used, the weight of the salt contained in the dosing solution is neglected and results in a corresponding error. This error was negligible in the tests involved herein, as the following considerations show:

As a usual case, the discharge of the turbines was about 1 550 cu. ft. per sec., and the rate of dosing was about 0.25 cu. ft. per sec., so that the ratio of dilution was about 1 to 6 200. The strength of the salt solution was about 25%, and its density about 1.18, so that the weight of salt per liter was $0.25 \times 1.18 = 0.295$, say, 0.3 of the weight of distilled water $= 0.3 \times 62.4 = 18.7$, say, 19 lb. per cu. ft. of salt solution, of which one-fourth, or, say, 5 lb., is the rate of salt injection per second. The total weight of water passing through the turbine is approximately

$$1\,550 \times 62.4 = 96\,700, \text{ say, } 100\,000 \text{ lb. per sec.}$$

Therefore, the error introduced by omitting the weight of the salt is, relatively, about $5 \div 100\,000 = 0.00005 = 0.005\%$, which is negligible for work of only 0.1 or even 0.01 of 1% precision.

75.—Error 12.—Sources and Sinks for Chemicals.—In the language of the mathematicians, there may be sources or sinks intermediate to the dosing and sampling stations, where chemical is either introduced into or lost from the water being measured. These gains or losses of chemical may be due to natural causes, such as mineral springs in the bed of the stream, or to artificial causes, such as result from the operations of industries along the channel.

If there is a mineral spring down stream from a leak or sink, where the flow of the spring and the discharge of the leak are about equal, the net result is an introduction of chemical between the two stations. In some such manner there might be a loss of chemical.

Where great accuracy is required, an investigation should be made to ascertain whether there are any such sources or sinks intermediate to the dosing and sampling stations. This is easily accomplished by taking samples, preferably simultaneously at the two stations. If serious discrepancies are found among the titrations of samples, appropriate

alterations in the scheme for measuring the flow of the water must be made.

In some locations it may be found necessary to test samples frequently to see whether there are changes in condition taking place from time to time, and this may have to be done during tests. Of course, it will usually be found possible to avoid this difficulty, but it should be carefully remembered when a discharge measurement is contemplated.

76.—Error 13.—Sources and Sinks for Water.—These are similar to those discussed in the preceding section, but are especially to be feared in the case of a turbine test. It is nearly always the case that there are leaks of more or less consequence through the concrete walls of penstocks and turbine pits. Besides this, in many cases the draft-tubes fail to discharge water throughout the entire cross-section. This results in eddies and negative currents which may introduce water containing more or less salt than should result from the salt injection intended for the test.

One of the easiest ways to examine this source of error is to shovel a considerable quantity of salt into the tail-race down stream from the tail-race sampling pipes. During the time that the salt is being shoveled into the tail-race the sampling pumps are kept in operation and samples of the tail-water are secured in the tail-race while samples of normal head-water are taken simultaneously in the head-race. If all samples titrate alike, it is safe to say that no tail-water was washed back into the tail-race during the test. A single test of this kind, however, may not be conclusive for all gate openings and turbine speeds. It may be necessary to repeat the test under a variety of conditions.

The best safeguard against this difficulty is to place the tail-sample pipes at a point in the system where the water is known to flow in only one direction throughout the entire cross-section. These pipes may even be placed in the head-race a short distance below the dosing pipes, but in this case, as the advantage of the mixing power of the turbine itself is lost, it will be necessary to design and operate the dosing system very carefully so as to secure as perfect a mixture as possible at the outset. See Section 56.

This method has been used with a fair degree of success in a case where the dosing solution was supplied to the distributing and sprinkling system by a centrifugal pump of insufficient capacity.

It is quite important in such cases to have a centrifugal pump and sprinkling system which will force the jets of water to a considerable distance laterally, in order to secure good mixtures.

77.—*Error 14.—Errors in Tables and Data.*—There may be some slight errors in the salt tables and diagrams (Table 1 and Plates XXXIII, XXXIV, and XXXV). These may be relatively large for the smaller values of the quantities tabulated. Thus, the theorem of Section 23, that a molecule of salt always displaces a portion of the water in which it is dissolved may need revision when more accurate data are available.

However, the results of the tests and tank calibrations described herein are of such a character that it may be concluded there was no error in respect to the accuracy of the tables which affects the results appreciably.

78.—*Error 15.—Theory.—Simplified Calculations.—Advantages of Measuring Samples by Weight.*—One of the best chances for error lies in the calculation itself. The simpler the calculations are made the less chance there is for errors. In Section 10 attention was directed to the system of weighing samples rather than measuring them by volume. In order to make the advantages of this method more apparent, the following calculation of the weight of the contents of the calibrator (November 25th, Table 35) is given, with a final reduction to volume for the purpose of checking the result determined in that table, 241.02 liters at 4° cent. The check is not absolutely perfect, owing principally to the fact that in computing the data for the calculation corresponding to that of the table, the weight, W'_2, has been taken at 860 grammes and the weight, W', at 10 grammes, instead of 1 003.6 and 11.671 grammes, respectively, which correspond more exactly to the figures of the table. Had these figures been taken, the ratio, F', would have been 85.99, instead of 86, and the result would have checked the figures of Table 35 very closely. The rounded weights have been chosen, as it is quite probable that the samples would be weighed in about that manner, it being a simple matter to adjust the quantity of solution to a given weight. It would be better practice, however, to adjust the weights so that all samples would titrate alike. Hence the titration, t, has been inserted, which represents the titration of 11.7 grammes of salt solution, this number being, aproximately, a multiple by ten of the density. The advantages of this method will undoubtedly

be appreciated after an examination of Table 41, which, with but few additional reductions, would include all the necessary data and the final calculations, thus displacing both Tables 34 and 35. These latter tables could be made somewhat simpler than they are, but, in the volumetric method, it is better to reduce all values to the temperature of the feed-water (or tail-water, which is at the same temperature as the head-water, within the limits of ordinary accuracy), so as to make the calculations consistent in appearance as well as in fact.

TABLE 41.—Illustrating the Advantages of the Method of Weighing Samples.

(The data have been computed from those of Tables 34 and 35 so as to represent closely the necessary data and calculations by the method of weights, Section 10, as applied to the calibration of the iron pipette, or calibrator, November 25th, 1914. All items marked *D* in the last column are parts of the data; all others result from computation. It will be seen that the temperature does not enter until it becomes necessary to pass to volumes.)

Explanation.	Line No.	Symbol.		Data.
Weight of salt solution taken for dilution with distilled water at temperature of laboratory, which temperature is not required	1	.....	11.7 grammes.	*D*
This salt solution is diluted with distilled water until the weight of the solution is	2	.....	1 000 grammes	*D*
Weight of dilution taken for titration	3	.....	10 grammes.	*D*
Mean titration of above samples	4	t	53.212 c. c.	*D*
Mean titration per kilogramme = 100 000 × Line 4 ÷ Line 1	5	v	454 800 c. c.	
Mean titration per 10-gramme sample of mixture from tank calibrator	6	t_2	53.273 c. c.	*D*
Mean titration per kilogramme of tail-water	7	v_2	5 327.3 c. c.	
Mean titration per 10-gramme sample of special dilution	8	t'_2	53.168 c. c.	*D*
Mean titration per kilogramme of special dilution	9	v'_2	5 316.8 c. c.	
Line 7 − Line 9	10	$v_2 - v'_2$	+ 10.5 c. c.	
Line 6 − Line 9	11	$v - v'_2$	449 491 c. c.	
Line 10 ÷ Line 11	12	y	0.0000234	
Weight of normal head water for special dilutions	13	W'_1	850 grammes.	*D*
Weight of salt solution for special dilutions	14	w'	10 grammes.	*D*
Ratio of dilution for special dilutions $[(W'_1 + w') \div w']$	15	F'	86.0	
Value of $(F' - 1)\, y = 85\, y$	16	$(F' - 1)\, y$	0.00199	
Value of the ratio of dilution in calibrator (by weights) found by dividing value of Line 15 by 1 + 0.00199 = F	17	F	85.829	
Weight of salt solution introduced into calibrator, which was then nearly full of normal canal water, and immediately afterward filled completely with normal canal water, the whole being agitated until the mixture was uniform	18	w	2.8137 kg.	*D*
W_2 = total weight of mixture in calibrator	19	$w\,F$	241.50	
Density of mixture at 4° cent.	20	d_2	1.0019	*D*
Volume at 4° cent. = $W_2 \div d_2$	21	Q_2	241.04	

It will be at once apparent that if it was the weight of the solution in the calibrator which was required, the calculations of Table 41 would be entirely independent of temperature observations, and therefore independent of temperature corrections. In the case of turbine tests, the only observation requiring a volumetric observation would be the determination of the weight of salt solution introduced into the head-water. This would probably best be done by observing the volume of salt solution consumed, exactly as was done during the tests described, with accurate weighings of standard volumes taken as samples from the sampling cocks in the 3-in. salt-solution supply pipe. These should be quickly measured and weighed on the spot and then poured back into the salt-solution tank. Several of these weighings should be made during the test, and the results should check well within the limit of permissible error. The mean of the results divided by the standard volume will be the density by which the total volume, or volume per second, is to be multiplied in order to get the total weight of solution consumed, or the weight per second, as the case may be.

The calculations of Table 41 are based on the last of Equations (27), Section 13, and the data have been reduced from those of Tables 34 and 35 by using Equations (38), Section 18.

79.—*Error 16.—Determination of Acting Head.*—It is the opinion of the writer that the determinations of acting head were the most doubtful of the data taken during the tests. The elevations of head- and tail-water could be determined quite accurately according to the requirements of the contract, but the real question at issue is as to the constant error involved in this method; and this leads to the more general question: How should head be measured for the purpose of turbine tests?

If one wishes to determine the energy actually entering the waterway for the turbine to be tested, it will be necessary to determine the actual kinetic energy of the water and the actual mean elevation of the water surface. If one wishes to determine the actual hydraulic efficiency of the installation—charging the wheel with the energy entering the waterway and deducting the energy which leaves the draft-tube—similar quantities for the tail-water must be determined. It would appear, however, that the latter operation should not be im-

posed by the terms of efficiency contracts, as it is urgently demanded that as little loss as possible result from tail-race flow.

Probably the contract should charge the turbine with all the energy flowing into the head-water passages and credit it with only such energy as it delivers at its shaft coupling. Contract efficiency and hydraulic efficiency, in the sense used here, are two entirely different things.

Now, it is highly probable that a sufficiently close estimate of the kinetic energy of the entering water can be computed from our practical knowledge of the character of flow of water in head-races, thereby avoiding the necessity for intricate current-meter or Pitot-tube surveys of water flow in such water passages.

Take, for example, the case of a head-race of width, $W = 2R$, when the water is flowing with parabolic distribution of velocities, looked on in plan, the axis of the parabola coinciding with the axis of the race. The total kinetic energy in the race would then be

$$E = \tfrac{1}{2}\int v^2\, dm = \frac{\gamma\, b}{2\, g}\int_{-R}^{+R}\left(V - \frac{y^2}{4\, a}\right)^3 dy = \frac{16}{35}\,\frac{\gamma\, b\, R\, V^3}{g}. \quad (143)$$

where $y^2 = 4\, ax$, $R^2 = 4\, aV$, $v = V - x$, $dm = \dfrac{\gamma\, v\, dA}{g}$, and $dA = b\, dy$, according to usual notation, $b\, W = A$ being the area of the cross-section of the race, b being the depth of water in the race, γ = specific weight of water, g = acceleration of gravity, and V being the maximum velocity in the section along its axis.

By a well-known property of the parabola, the mean velocity, V_m, in this problem, must be two-thirds of the maximum velocity, that is, $\frac{2}{3}\, V = V_m$. Hence, the discharge, Q, is equal to $A\, V_m$, the weight of the discharge is $\gamma\, A\, V_m$, and the total energy is

$$E = \frac{54}{35}\,\frac{\gamma\, b\, R\, V_m^3}{g} = \frac{27}{35}\,\frac{\gamma\, A\, V_m^3}{g}.$$

Consequently, as the mean velocity head is equal to the total energy divided by the weight of the discharge, we must have

$$\left.\begin{array}{l}\text{Mean velocity head for parabolic distribution in horizontal planes}\end{array}\right\} = \frac{54}{35}\,\frac{V_m^2}{2\, g} = 1.54\,\frac{V_m^2}{2\, g}. \quad (144)$$

It is quite apparent, therefore, that for anything like ordinary distribution of velocities, the mean velocity head is greater than the head due to the mean velocity in the raceway.

If we take the case of paraboloidal distribution in a circular race, it may be shown that

$$\left.\begin{array}{l}\text{Mean velocity head for paraboloidal}\\ \text{distribution in circular races run-}\\ \text{ning full}\end{array}\right\} = 2\,\frac{V_m^2}{2\,g} \quad \ldots\ldots (145)$$

The case of triangular distribution in horizontal planes in a rectangular race is identical with paraboloidal distribution in circular races running full, in so far as the relation between mean velocity head and head of mean velocity is concerned, that is, in either case the mean velocity head is twice the head due to the mean velocity.

At first sight, these last results do not seem to agree with parabolic distribution, in that the latter would appear to have relatively more high velocities than triangular distribution, and the numerator of mean velocity head is made up of cubes of the elementary velocities while the denominator contains first powers only. Apparently, the coefficient should be higher for parabolic than for paraboloidal or triangular distribution. The deception lies in the fact that V_m is greater relatively to V in parabolic distribution than it is in the other cases. Thus the actual mean velocity heads for a given maximum velocity, V, in the two cases are:

$$\left.\begin{array}{ll}\text{Parabolic distribution} & = 0.686\,\dfrac{V^2}{2\,g}\\ \text{Paraboloidal or triangular distribution} & = 0.50\,\dfrac{V^2}{2\,g}\end{array}\right\} \quad \ldots (146)$$

Thus, for a given maximum velocity, the mean velocity head is greater for parabolic than for paraboloidal or triangular flow, and the converse is true for a given mean velocity.

80.—Actual Distribution of Velocities in Raceways.—The foregoing assumptions are not sufficiently exact to represent accurately the actual conditions of flow in raceways. It is much more likely that the flow would approximate the conditions which may be indicated by adding a constant velocity, V_1, to the velocities of parabolic distribution in horizontal planes. Thus, if $S = V + V_1$ be substituted for V in the foregoing integral, Equation (143), V_1 being constant,

$$E = \frac{1}{2}\int v^2\,d\,m = 2\,\frac{\gamma\ b\ \sqrt{a}}{g}\int_0^V (S-x)^3\,d\,(x^{\frac{1}{2}})$$

therefore,

$$E = \frac{2\,\gamma\,b\,\sqrt{a}}{g}\left(k^3 - k^2 + \frac{3}{5}k - \frac{1}{7}\right)V^{\frac{7}{2}} \quad \ldots\ldots(147)$$

where $k = S \div V = (V + V_1) \div V$ and $y^2 = 4\,ax$.

By noting that $2\,\sqrt{a}\,V^{\frac{1}{2}} = R$, and $2\,R = W$, Equation (147) may be written

$$E = \frac{\gamma\,b\,W\,V^3}{2\,g}\left(k^3 - k^2 + \frac{3}{5}\,k - \frac{1}{7}\right) \quad \ldots\ldots(148)$$

Moreover, if V_m is the mean velocity in the raceway,

$$V_m = \frac{2}{3}V + V_1 = S - \frac{V}{3} = \left(k - \frac{1}{3}\right)V \quad \ldots\ldots(149)$$

and the discharge is

$$Q = b\,W\,V_m = \left(k - \frac{1}{3}\right)b\,W\,V \quad \ldots\ldots(150)$$

from which the mean velocity head is evidently

$$\frac{E}{Q\,\gamma} = \frac{\left(k^3 - k^2 + \frac{3}{5}k - \frac{1}{7}\right)}{\left(k - \frac{1}{3}\right)}\,\frac{V^2}{2\,g} \quad \ldots\ldots(151)$$

But, from Equation (149),

$$V^2 = \frac{V_m^2}{\left(k - \frac{1}{3}\right)^2} \quad \ldots\ldots(152)$$

so that, in terms of V_m, mean velocity head

$$= \frac{k^3 - k^2 + \frac{3}{5}\,k - \frac{1}{7}}{\left(k - \frac{1}{3}\right)^3}\,\frac{V_m^2}{2\,g} \quad \ldots\ldots(153)$$

This equation will approximate more or less closely the actual mean velocity head when the value of k approximates the actual value, based on the observed ratio of S to V.

Take, for example, the actual distribution in the tail-races observed by the writer in 1911.* In the first of these two tables, $S \div V = \frac{306}{306 - 150} = 1.96$. In the second, $S \div V = \frac{311}{145} = 2.14$,

* *Transactions*, Am. Soc. C. E., Vol. LXXVI, pp. 827-828, Tables 3 and 4.

the mean being 2.05, or approximately 2. Again, in Test *G*, of the present turbine tests, the corrected current-meter observations furnish the distribution shown in Table 42.

TABLE 42.—VELOCITIES AT ONE HUNDRED UNIFORMLY DISTRIBUTED METER POINTS IN HEAD-RACE OF UNIT NO. 13, TEST *G*.

(The unit is $\frac{1}{100}$ ft. per sec., except in the final averages, where it is 1 ft. per sec.)

Depths.	Velocities.									
1	260	261	286	256	261	259	257	286	317	351
2	301	294	292	265	263	253	267	268	353	323
3	366	300	300	287	278	258	268	259	294	272
4	304	293	302	261	254	266	254	268	300	312
5	360	318	297	270	249	260	258	279	297	306
6	338	298	263	256	269	253	275	262	296	302
7	325	271	256	259	276	238	278	308	267	304
8	265	215	256	250	234	238	291	295	267	248
9	244	261	266	269	227	230	270	307	263	199
10	156	218	211	208	211	225	248	310	350	000
	2 919	2 729	2 729	2 581	2 522	2 480	2 666	2 842	3 006	2 619

Total.....270.93

V_m = mean velocity = 2.7093.

V^2_m = 19.8871, say, 19.89.

In this distribution there are two extremely low velocities which probably should be neglected. Perhaps a velocity of 2 would be a fair minimum, so that $k = 366 \div (3.66 - 2) = 2.2$. It might be assumed, therefore, that:

In tail-races of 1911 tests, $k = 2.0$;
In head-races of 1914 tests, $k = 2.2$;

the corresponding values of the mean velocity head, by Equation (153), being:

$$\left.\begin{aligned} &\text{Mean velocity head in tail-race, tests of 1911.....} \\ &\text{Mean velocity head in head-race, tests of 1914.....} \end{aligned}\right\} \begin{aligned} &= \frac{4\,779}{4\,375}\frac{V^2_m}{2g} = 1.09\,\frac{V^2_m}{2g} \\ &= \frac{699}{655}\frac{V^2_m}{2g} = 1.07\,\frac{V^2_m}{2g} \end{aligned} \Bigg\} \quad ..(154)$$

Consequently, it may be said that the mean velocity head in the tail-races, down stream from the stilling racks, in the tests of 1911,

on a horizontal unit driven by a string of six turbines, was nearly the same relative to the head due to mean velocity as in the head-races, up stream from the trash racks, in the tests of 1914, on a vertical double-runner unit discharging nearly as much water as the turbine unit on the former occasion.

In the case where the maximum is about twice the minimum velocity in a raceway, it may be said, therefore, that, as an average,

$$\left.\begin{array}{l}\text{Mean velocity head, where } k = 2 \text{ and} \\ V_m = \text{mean velocity in raceway} \ldots\end{array}\right\} = 1.09\,\frac{V^2{}_m}{2\,g} \ldots . (155)$$

81.—Mean Velocity Head During the Tests.—We are now in a position to compute the actual mean velocity head in the head-races during any of the tests. The elevation of the floor of the head-race was 176 ft. above sea level. In Test *G* the water surface was at Elevation 199, the depth, therefore, being 23 ft. The width of the race is about 27 ft., so that the area of section was about 621 sq. ft. The discharge for this test was about 1 662 cu. ft. per sec., which corresponds to a mean velocity of 1 662 ÷ 621 = 2.68 ft. per sec. The head due to this velocity is 0.112 ft., which makes the mean velocity head = 0.12 ft.

Test 41 was taken very near the point of maximum efficiency. The discharge was 1 520 cu. ft. per sec., the elevation of the head-water was 196.7, the depth, consequently, was 20.7 and the area 560 sq. ft. Thus, the head due to the mean velocity was 0.115 ft., and the mean velocity head, by Equation (155), was 0.125 ft.

In some of the tests the elevation of the head-water was observed at the crown of the arch, at the point where the water changes its flow to descend to the wheel. At this point the water piled up, by reason of the change in direction, the surface water thereby coming very nearly to rest. Theoretically, this should equal the velocity head of the water before it is retarded. Table 43 gives the results of such observations as were made during the tests; and it will be seen that the rise of water due to its stoppage approximates closely the rise computed by the equations of the foregoing theoretical and practical study deduced in another way.

82.—Actual Mean Velocity Head.—Regardless of all theory, one may observe the individual velocities at the various points in a cross-section, and compute the velocity head at each point, thus enabling a

determination of the mean velocity head from direct calculations. This procedure would be indicated by the following equation, which is self-explanatory:

$$\left.\begin{aligned}\text{Mean velocity head in raceway, where } V_m \\ = \text{the mean velocity}\end{aligned}\right\} = \frac{E}{\gamma Q} = \frac{\frac{\gamma}{2g}\Sigma v^2 d Q}{\gamma Q} = \frac{1}{2g}\frac{\Sigma v^3 d A}{A V_m} = \frac{\frac{d A}{A}\Sigma v^3}{V^3_m}\frac{V^2_m}{2g} \quad \ldots (156)$$

In words, the theorem runs:

The ratio of the mean velocity head of flowing water to the head due to its mean velocity is equal to the ratio of the mean of the cubes of the velocities in the cross-section to the cube of the mean velocity.

Therefore,

$$\frac{k^3 - k^2 + \frac{3}{5}k - \frac{1}{7}}{\left(k - \frac{1}{3}\right)^3} = \frac{\frac{d A}{A}\Sigma v^3}{V^3_m} \quad \ldots (157)$$

for the distribution of velocities considered in Section 80.

TABLE 43.—Differences in Elevation Between Moving and Still Water in Head-Races Observed During Certain of the Tests on Units Nos. 11 and 12.

(The unit is 1 ft.)

Unit No. 11.				Unit No. 12.			
Test.	Average elevation of head-water by slanting rod.	Average elevation of head-water by rods and boards.	Difference in elevation.	Test.	Average elevation of head-water by slanting rod.	Average elevation of head-water by rods and boards.	Difference in elevation.
111	195.46	195.29	0.17	200	195.93	195.68	0.25
112	194.80	194.63	0.17	201	196.12	195.92	0.20
113	195.84	195.67	0.17	202	195.50	195.33	0.17
114	195.26	195.09	0.17	203	195.72	195.51	0.21
115	194.88	194.72	0.16	204	195.98	195.80	0.18
116	195.98	195.79	0.19	205	195.31	195.19	0.12
117	196.87	196.67	0.20	206	195.10	194.98	0.12
118	197.51?	197.19	0.32?	207	195.25	195.12	0.13
119	195.11	194.94	0.17	208	195.33	195.18	0.15
120	196.04	195.88	0.21	209	195.75	195.66	0.09
121	195.12	195.00	0.12	210	195.97	195.82	0.15
122	195.78	195.67	0.11	211	196.52	196.34	0.18
				212	194.94	194.86	0.08

when the vertical projection of the velocities on a horizontal plane may be represented by a rectangle capped by a segment of a parabola, the velocities themselves being supposed to be horizontal and equal in any vertical, and k being the ratio of the combined height of the rectangle and segment (maximum velocity) to the height of the segment.

Applying Equation (156) to the cases of the tests of 1911 and 1914, by computing the mean of the cubes of the velocities in the tables and dividing by the cube of the mean velocities, the following may be determined as the actual coefficients for the head due to mean velocity:

$$\left.\begin{aligned} \text{Tests of 1911, coefficient} &= \frac{14.5}{13.5} = 1.07 \\ \text{“ “ 1914, “ “} &= \frac{21.30}{19.89} = 1.07 \end{aligned}\right\} \quad \ldots\ldots (158)$$

The values agree very closely with those deduced above, based on less direct assumptions. The general conclusion is that Equation (157) gives the value of the coefficient in terms of k with a considerable degree of precision, where the flow is not unusual in character.

83.—Personal Errors and Carelessness.—Shaking Flasks.—It requires experience for a person to develop what is technically called a personal error. Such an error is a systematic error in that it always occurs in the same sense and in about the same degree under given circumstances. In many cases this error may be eliminated by properly arranging the methods of observing. For example, if a chemist has acquired by experience the fixed habit of reading a burette in such a manner that all observations are too high by 0.02 c.c., then the error is eliminated when he subtracts the lower from the higher reading to determine the number of cubic centimeters of silver nitrate consumed in a titration. Hence, if it requires two gauge readers to determine the acting head during a turbine test, it would be advisable to have them change stations during the test, so as to have equal numbers of head and tail-gauge readings by each observer. Again, in the case of pipettings, two chemists may handle their instruments in different ways, so that pipettings by one of them will always be slightly larger than by the other. Such an error may be eliminated from discharge measurements by the method of group titrations, discussed in Section 48.

Carelessness, however, can be guarded against only by its prevention. Ignorance is closely akin to carelessness, and must be prevented

by having every observer understand exactly what he is to do and how it is to be done. The rules must be specific, pertinent, and concise. Therefore, the person who undertakes to conduct a chemical test must have every detail of it at his command. He must go over the ground thoroughly in advance, and provide for every contingency.

Nothing illustrates the importance of these remarks better than the difficulties encountered in securing accurate chemical work during the first few of the turbine tests described herein. It was apparent that something was radically wrong. After the first week's work, the writer devoted an entire day in the laboratory to repeating some work which was manifestly in error, the result being the discovery that the dilutions were not being thoroughly mixed by sufficient agitation of the flasks containing them.

All chemists know that flasks must be shaken in order to secure perfect mixtures, but it does not seem that there is any definite rule as to how many times, or how vigorously, the shaking must be administered. It would seem that this should depend on the shape and size of the receptacle in which the mixture is to be effected and the manner of shaking.

In the case of an ordinary stoppered liter flask with a long neck, about 0.6 in. in diameter, it requires from 25 to 30 double shakes or inversions, with time for the air to rise to the surface of the liquid in each position, in order to secure the uniform mixture of 10 c.c. of nearly saturated salt solution with about 990 c.c. of distilled water. In order to insure results, the rule was adopted that such operations should require 40 double inversions, with sufficient time allowance in each position for the air to rise to the surface. In the case of a different size or shape, the rule will probably require modification.

The calibrations of the flasks are given in diagrammatic form in Fig. 6.

84.—Effect of Variations in Discharge on the Concentration of Tail-Water Samples.—The discharge of a turbine is likely to vary, even when efforts are made to maintain uniform conditions during a test. It is necessary, therefore, to show that the method of testing and sampling does not introduce an error.

First, it is necessary to show that, when a tail-race sample is pumped into a sample bottle at a uniform rate, the resulting concentration is the time-average concentration of the water at the sampling pipe, in-

stead of the mean concentration relative to all the volume of the water passing the pipe.

The time-average concentration is evidently

$$\int \frac{c_2\, d\, t}{T} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (159)$$

and the total quantity of salt in the sample is

$$\int c_2\, x\, d\, t \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (160)$$

where x is the rate at which the sample is pumped into the sample bottle, which rate is supposed to be constant. The total volume of the sample is

$$\int x\, d\, t \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (161)$$

CURVES SHOWING VOLUMES OF FLASKS AT VARIOUS TEMPERATURES,

As Derived from Direct Volumetric Comparisons with the Contents of Flask No. 1456 or Flask No. 11, which were Calibrated by the U.S. Bureau of Standards.

Note:-Flask No. 1456 was calibrated by Arthur Schweier at the Chemical Laboratory of the University of Pittsburgh, and its Contents at 19.7°C. were found to be 1002.68 c.c., as compared with 1003.00 c.c. given by the U.S.B.S.

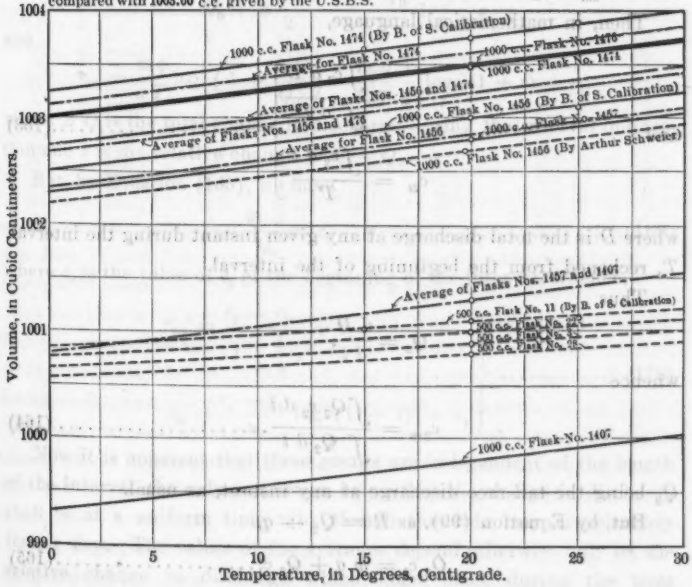


FIG. 6.

Therefore, the concentration of the sample is

$$c_2 = \frac{\int c_2 x\, d\, t}{\int x\, d\, t} = \frac{\int c_2\, d\, t}{T} \quad \ldots\ldots\ldots\ldots\ldots\ldots (162)$$

as x is constant.

By Equation (159), however, this is a time average. Hence, the proposition is proved that the concentrations of samples pumped at uniform rates are time averages.

We may now investigate the relations existing between time averages and volume averages, with especial reference to tail-water concentrations.

Let c_{2m} be the average concentration with reference to the total volume of discharge during a given time, T, which is supposed to be an interval during which the discharge, Q_2, varies, and which is included as part of the time of a test. Let c_{2t} be the time average of the concentration during the same period. The object is to find the relation of c_{2m} to c_{2t}.

Then, in mathematical language,

$$\left.\begin{aligned} c_{2m} &= \frac{\int c_2\, d\, D}{\int d\, D} \\ \text{and} \qquad c_{2t} &= \frac{\int c_2\, d\, t}{T} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (163)$$

where D is the total discharge at any given instant during the interval, T, reckoned from the beginning of the interval.

Thus

$$Q_2 = \frac{d\, D}{d\, t},$$

whence

$$c_{2m} = \frac{\int Q_2\, c_2\, d\, t}{\int Q_2\, d\, t} \quad \ldots\ldots\ldots\ldots\ldots\ldots (164)$$

Q_2 being the tail-race discharge at any instant, as usual.

But, by Equation (99), as $R = Q_2 \div q$,

$$Q_2\, c_2 = m\, q + Q_2\, c_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots (165)$$

where $m = c - k\, c_1$.

Therefore, by Equations (162), (163), and (164),

$$\left.\begin{aligned} c_{2m} &= \frac{\int (m\,q + Q_2\,c_1)\,d\,t}{\int Q_2\,d\,t} = \frac{m\,q\,T}{\int Q_2\,d\,t} + c_1 \\ c_{2t} &= \frac{\int \frac{m\,q + Q_2\,c_1}{Q_2}\,d\,t}{T} = m\,q \int \frac{d\,t}{Q_2} + c_1 \end{aligned}\right\} \quad \ldots\ldots (166)$$

From the nature of the integrals, it is apparent that the discrepancy between c_{2m} and c_{2t} depends on the manner in which the discharge varies during the interval over which the averages are to be computed. Suppose, for example, that

$$Q_2 = Q_0 + a\,t \quad \ldots\ldots (167)$$

where a is constant and Q_0 is the discharge at the beginning of the interval, t being the time reckoned from that instant.

Then we may show that

$$c_{2m} = \frac{m\,q}{Q_0 + a\,\frac{T}{2}} + c_1 = \frac{m\,q}{Q_0}\,\frac{1}{1 + \frac{1}{2}\,z} + c_1 \quad \ldots\ldots (168)$$

and

$$c_{2t} = \frac{m\,q}{a\,T} \log.\left(1 + \frac{\Delta\,Q}{Q_0}\right) = \frac{m\,q}{Q_0}\,\frac{1}{z} \log.\,(1 + z) + c_1$$

where $\Delta\,Q$ is the total change in discharge during the interval in question and z is the relative change, $\Delta\,Q \div Q_0$.

But, by Equation (165), we have

$$\frac{m\,q}{Q_0} = c_0 - c_1 \quad \ldots\ldots (169)$$

where c_0 is the value of c_2 at the beginning of the interval, whence,

$$\left.\begin{aligned} c_{2m} &= \frac{c_0 - c_1}{1 + \frac{1}{2}\,z} + c_1 \\ c_{2t} &= \frac{c_0 - c_1}{z} \log.\,(1 + z) + c_1 \end{aligned}\right\} \quad \ldots\ldots (170)$$

Now it is apparent that these results are independent of the length of the interval, the only restriction being that the change in discharge shall be at a uniform time rate, though this rate may be relatively fast or slow. The values of the averages depend otherwise only on the relative change in discharge which takes place during the time considered.

Let it be supposed, therefore, that the relative changes of discharge are, respectively, 0.1, 0.2, 0.3, 0.4, and 0.5, first in the sense of increase and then the reverse. The resulting comparisons of the values of v_{2m} and v_{2t} are given in Table 44, together with the calculations.

TABLE 44.—CALCULATION OF THE RATIOS OF THE TIME AVERAGE CONCENTRATIONS IN THE TAIL-RACE TO THE CORRESPONDING MEAN CONCENTRATIONS FOR ANY INTERVAL OF TIME DURING WHICH THE DISCHARGE INCREASES OR DECREASES AT A UNIFORM RATE EXPRESSED AS A FUNCTION OF THE RELATIVE CHANGE IN DISCHARGE DURING THE INTERVAL. THE CONCENTRATION OF THE TAIL-WATER AT THE BEGINNING OF THE INTERVAL IS ASSUMED TO BE THE EQUIVALENT OF 110 C.C. OF SILVER NITRATE TITRATION PER LITER, AND THE NORMAL HEAD-WATER CONCENTRATION 25 C.C.

(Thus $v_0 - v_1 = 85$ c.c.)

z	$\frac{1}{z}\log.(1+z)$	$\frac{1}{1+\frac{1}{2}z}$	v_{2t} c.c.	v_{2m} c.c.	$v_{2t} - v_{2m}$ c.c.	$\frac{v_{2t}}{v_{2m}}$	Percentage of error.
0.5	0.8109	0.8000	93.93	93.00	0.93	1.0100	1.0
0.4	0.8412	0.8333	96.50	95.83	0.67	1.0070	0.70
0.3	0.8745	0.8696	99.33	98.92	0.41	1.0041	0.41
0.2	0.9116	0.9091	102.49	102.27	0.22	1.0022	0.22
0.1	0.9531	0.9524	106.01	105.95	0.06	1.0006	0.06
0.05	0.9758	0.9756	107.94	107.93	0.01	1.0001	0.01
0.00	1.0000	1.0000	110.00	110.00	0.00	1.0000	0.00
−0.05	1.0258	1.0256	112.19	112.18	0.01	1.0001	0.01
−0.1	1.0540	1.0526	114.59	114.47	0.12	1.0008	0.08
−0.2	1.1155	1.1111	119.82	119.44	0.38	1.0032	0.32
−0.3	1.1890	1.1765	126.06	125.00	1.06	1.0085	0.85
−0.4	1.2770	1.2500	133.55	131.25	2.30	1.0175	1.75
−0.5	1.3862	1.3333	142.83	138.33	4.50	1.0325	3.25

It is very evident from the array of figures in Table 44 that any change in discharge during a test causes an over-estimate of concentration of tail-water where the samples are taken at a uniform rate, but that the error introduced is less than 0.1% for variations as great as 10% of the discharge, and less than 0.01% for variations less than 5% of the discharge. Moreover, these errors will be greatly diminished when the power is constant during a part of the test, so that they are entirely negligible in the tests now being examined, as the maximum variation of power during any of the decisive tests was less than 5 per cent.

The general truth, that time averages of concentrations for variable discharges with uniform dosing are too large as compared to volume averages, may be seen without recourse to computations.

First, suppose the discharge to increase during a certain interval of time. Then the concentration must decrease during that interval. Now, a time average gives equal weights to the momentary concentrations at all instants, and a volume average, in this case, gives greater weight to the momentary concentrations as time progresses. Therefore, for increasing discharge, the volume average gives greater weights to the weaker concentrations and results in a smaller value than does a time average.

Second, suppose the discharge to decrease during a certain interval of time. Then the concentration must increase during that interval; but the time average gives equal weights to the momentary concentrations at all instants, and a volume average, in this case, gives less weight to the momentary concentrations as time progresses. Therefore, for decreasing discharge, the volume average gives smaller weights to the stronger concentrations, and results in a smaller value than does a time average.

Therefore, for variable discharges, with uniform dosing, the time average is always greater than the volume-average concentration.

This truth may be seen very clearly in still another manner. Suppose two tail-water samples are being accumulated in two sample bottles, the first being pumped into its bottle at a uniform rate while the second is pumped into its bottle at a rate proportional to the discharge, both samples being started at the same rate.

Then, if the discharge is increasing, it is clear that the first bottle is not receiving enough of the weaker solutions as time progresses; but, if the discharge is decreasing, it is clear that the first bottle is receiving too much of the stronger solutions as time progresses.

It follows, in either case, that the concentration is greater in the first than in the second sample bottle.

85.—Securing Volume Average Concentrations.—In determining on the method of sampling with small pumps drawing their supply from suctions in the tail-race, it was realized that time averages of concentration would result, rather than volume averages; but it was thought that it would be better to have something which was definite than to try to secure a volume average in the absence of any method for pumping, or otherwise taking, samples from the tail-race at rates which varied in direct proportion to the velocity of the tail-water at the inlet of suctions.

In considering the various methods of taking continuous samples, both the vacuum and gravity methods were discussed. The power-house was equipped with a vacuum system, the vacuum being derived from one or more of the draft-tubes. The samples might have been sucked up by this vacuum and tapped off at various points by a simple device. An inverted caisson might have been constructed and pumped out, or the sump well in the power-house might have been used, in either of which a system of pipes discharging the samples continuously by gravity into the sample bottles could have been installed.

It was thought, however, that the small pumps would be more reliable as to the constancy of rate than either of these methods, and they were therefore adopted.

If it is desired to secure volume-average samples, both as to time and place, in the tail-race, the object may be accomplished by installing the suctions of the pumps in the form of Pitot tubes with static orifices attached, the Pitot tubes pointing up stream with their axes parallel to the center line of the race.

Both the Pitot tubes and static orifices must be connected by small tubes leading from holes in their walls to the static and dynamic gauges arranged alongside the corresponding pumps, or other devices for drawing out the samples.

An attendant regulates the rate of sampling at each pump or sampling device, so that the dynamic and static columns, which are drawn up in the usual manner on the graduated scales corresponding to any pump, register the same elevation.

If this condition is maintained with reasonable exactness during a test, the concentration of each sample will be a volume average for that sample during the test.

If the relative volumes of the samples which have been taken in this manner are used as weights for the corresponding samples, the weighted mean of all the samples will be the volume-average concentration for the test as a whole.

Finally, if all the samples thus taken are thoroughly mixed in one receptacle, or if given fractions of each of the samples are thus mixed, the concentration of the mixture will be the volume-average concentration for the test.

It may be added that similar procedure will serve for the method of weights, as against the method of volumes.

86.—Comparison of Discharges by the Method of Special Dilutions with Those Simultaneously Determined by the Method of Unbalanced Evaporations.—In these studies it has been the endeavor to develop a method which will eliminate as far as possible all the errors of the chemistry, as well as to discover the corrections to be applied in the chemistry, where the method of unbalanced evaporations has been used.

Unbalanced evaporations result in the methods connected with such equations as (14), (17), (46), (53), (98), and (99). The corrections have been discussed in Section 43, *et seq.* The method of special dilutions applies particularly to such equations as (27), (28), (39), (44), (45), (56), (106), (107), and (108). It will strengthen the confidence in the method of correcting the errors for unbalanced evaporations and in the method of special dilutions, or balanced evaporations, if the results by these two methods agree.

In order to facilitate the comparison, Table 45 has been prepared. This not only enables a direct comparison in the individual tests, but also enables the direct comparison of the aggregates of discharge for all the tests on each unit, and all the tests collectively.

TABLE 45.—Comparison of Discharges Obtained by the Method of Special Dilutions with Those by the Method of Unbalanced Evaporations.

Unit No. 11.			Unit No. 12.			Unit No. 13.		
Test No.	Discharge by special dilution.	Discharge by unbalanced evaporation.	Test No.	Discharge by special dilution.	Discharge by unbalanced evaporation.	Test No.	Discharge by special dilution.	Discharge by unbalanced evaporation.
105	1 443.4	1 448.8	200	1 564.5	1 571.0	23 *B*	1 398.9	1 421.8
106	1 539.0	1 538.5	201	1 515.7	1 515.0	26 *B*	1 497.4	1 474.1
107	1 519.4	1 518.9	202	1 481.1	1 470.1			
108	1 487.7	1 475.8	203	1 517.8	1 520.6			
109	1 492.0	1 482.4	204	1 505.3	1 505.0			
			205	1 512.2	1 511.0			
			206	1 491.4	1 493.5			
			207	1 485.4	1 488.0			
			208	1 501.8	1 496.2			
			209	1 560.0	1 561.7			
			210	1 539.4	1 543.2			
			211	1 544.1	1 545.9			
			212	1 489.4	1 492.9			
Sub-totals.	7 481.5	7 464.4	.....	19 708.1	19 714.1	.....	2 896.3	2 895.9

The aggregate discharge may be summarized as follows:

AGGREGATE DISCHARGE.

Unit No.	Special dilutions.	Unbalanced evaporations.
11	7 481.5	7 464.4
12	19 708.1	19 714.1
13	2 896.3	2 895.9
Totals.....	30 085.9	30 074.4

The tests which are compared for Unit No. 11 were the first five which were run on this unit, and they were not so good as the later tests. No unbalanced evaporations, however, were made in the later tests, so that only five tests are available for the purpose of comparison. The discrepancy between the results by the two methods, nevertheless, is only 17.1 cu. ft. per sec. in about 7 500, or about $\frac{1}{4}$ of 1%, for the tests on Unit No. 11.

If the tests on Units Nos. 12 and 13 are compared collectively, the result is as follows:

AGGREGATE DISCHARGES OF UNITS NOS. 12 AND 13.

Special Dilutions.	Unbalanced Evaporations.
22 604.4	22 610.0

The difference is 6.4 cu. ft. per sec. in about 23 000, or a discrepancy of less than 0.03 per cent. In the case of the aggregates for all the tests, where both methods were used, the discrepancy is only slightly more than 0.03 per cent.

It may be concluded, therefore, that the method of correcting titrations in the case of unbalanced evaporations and the method of special dilutions are theoretically and practically correct.

87.—Determination of Size of Solution Tank, Sizes of Samples, and Strength of Silver Nitrate Solution.—In laying out a series of tests, it will be convenient to have simple formulas embracing the relations of titrations, volumes, and sizes of samples. As a general rule, it will be on the side of economy, as regards the capacity of the solution tank, to have the strength of the dosing solution as great as practicable. It was found quite satisfactory to have a salt solution of 300 grammes per liter, but as any increase in strength above this value adds considerably to the difficulty of dissolving the additional quantity of salt

required, it will generally be best to adhere to 300 grammes per liter, or less, as the occasion may seem to indicate.

As regards the concentration of the silver nitrate, it should be observed that the stronger this solution the more salt it will require to create a given titration on a sample of given size. Therefore, for economy of salt, it will be desirable to keep the nitrate solution as weak as consistent with definiteness of end points.

It follows from these remarks that computations involving the concentrations of the salt solution and silver nitrate should always have regard to the possible and practicable values of these quantities. The quantities which enter the equations are so related that there will be found a great variety of ways in which the tests may be planned, and this will be found very satisfactory, as it is rare that two series of tests on different installations will demand the same treatment.

Let e = salt equivalent of 1 gramme of silver nitrate = 0.344;

c = concentration of salt solution;

c_s = concentration of silver nitrate solution;

ρ = ratio of dilution of salt solution with distilled water;

t_1 = titration of a sample of normal head-water;

t_2 = titration of a sample of tail-water or special dilution;

V = nominal volume of a sample for titration after evaporation, or of dilute salt solution as pipetted into its casserole, that is, $V = V_1 = V_2 = V'_2$;

ρ_2 = ratio of sample volume measured for evaporation to the nominal volume to which the evaporation is carried before the titration; this also applies to ρ'_2;

R = ratio of dilution anticipated in any test;

v = titration per liter of salt solution based on the titration of a sample of dilute salt solution of the volume, V;

t = actual titration of the sample of dilute salt solution of the volume, V;

S = measured volume of a sample of tail-water, special dilution, or computed volume of a sample of normal head-water; that is, $S = S_1 = S_2 = S'_2$.

Then, the following relations are obvious:

$$e = 0.3441 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (171)$$

$$v = \frac{1\,000\,c}{c_s\,e} = \frac{2\,906\,c}{c_s} \quad \ldots\ldots\ldots (v \text{ in cubic centimeters}) \ldots . (172)$$

Therefore, if c is the concentration of a sample of any kind, and t the titration, S being the volume of the sample, in liters, we have:

$$t = \frac{2\,906\,S\,c}{c_s} \quad \text{(general formula for titration)} \quad \ldots\ldots(173)$$

Again,

$$v = \frac{\rho\,t}{V} \quad \ldots\ldots(174)$$

so that

$$\frac{c}{c_s} = \frac{v}{2\,906} = \frac{\rho\,t}{2\,906\,V} = \frac{\rho\,\rho_2\,t}{2\,906\,S} \quad \ldots\ldots(175)$$

$$c_s = \frac{871\,800\,V}{\rho\,t} = \frac{871\,800\,S}{\rho\,\rho_2\,t} \quad (c = 300) \quad \ldots\ldots(176)$$

From Equations (99),

$$R = \frac{v - k\,v_1}{v_2 - v_1} \quad \ldots\ldots(177)$$

or

$$R = \frac{v}{v_2 - v_1} \quad \text{(approximately)} \quad \ldots\ldots(178)$$

as v_1 is supposed to be small, relatively to v. Thus:

$$R = \frac{\rho\,\frac{t}{V}}{\frac{t_2}{S_2} - \frac{t_1}{S_1}} \quad \text{(approximately)} \quad \ldots\ldots(179)$$

but $V = V_1 = V'_2$, $S = S_1 = S_2$, $t = t_2 = t'_2$, etc., so that

$$R = \frac{\rho\,\rho_2\,t}{t_2 - t_1} \quad \ldots\ldots(180)$$

or,

$$\rho\,\rho_2 = R\left(1 - \frac{t_1}{t_2}\right) \quad \ldots\ldots(181)$$

From this and an equation of the form of Equation (173),

$$\rho\,\rho_2 = R\left(1 - \frac{2\,906\,S_1\,c_1}{c_s\,t_2}\right) \quad \ldots\ldots(182)$$

or

$$c_s = \frac{2\,906\,S_1\,c_1}{t_2\left(1 - \frac{\rho\,\rho_2}{R}\right)} \quad \ldots\ldots(183)$$

which, when multiplied by Equation (175), gives

$$c = \frac{\rho\,\rho_2\,c_1}{1 - \frac{\rho\,\rho_2}{R}} = \frac{R\,c_1}{\frac{R}{\rho\,\rho_2} - 1} \quad \ldots\ldots(184)$$

The foregoing equations, of course, assume that the method of special dilutions, or balanced evaporations and titrations, is to be used.

No normal head-water is evaporated, but the planning of the tests requires that the properties of the normal head-water shall be taken into account, so that, in any case, a series of analyses of the head-water should be made to lead to an accurate balance of evaporations in the actual tests.

The value of c_1, therefore, will be known in advance, having been determined from analyses made under circumstances closely simulating an actual test, and the value of S_1 may be chosen equal to S_2 in the foregoing equations. It is not likely that c_1 will be so variable as to render a close balance of titrations impracticable. The object of the calculations will not be frustrated by considerable relative variations in c_1, when they do occur.

Let it be required, for example, to plan a series of tests where the discharge will vary from 1 000 to 1 500 cu. ft. per sec. It may be observed that the ratio of dilution, R, and therefore, R', can be closely the same in all the tests by varying the rate of discharge, q, of salt solution so as to maintain

$$\frac{Q}{q} = R \text{ constant} \ldots\ldots\ldots\ldots\ldots\ldots (185)$$

The values of $t_2 = t'_2 = t$, $S_2 = S'_2$, $V_2 = V'_2 = V$, and ρ, may be chosen arbitrarily, from which the values of c and c_s may be computed so as to satisfy all the requirements, it being merely necessary to see that c and c_s do not take on impractical values. When they do, a different choice of dimensions, capacities, titrations, or rates of dilution must be made, and repeated if necessary until satisfactory proportions are finally determined.

Therefore, let the data be taken as follows:

Evaporations are to be of ½-liter samples to 10 c.c. in a casserole for titration, or $S = \frac{1}{2}$, $V = 0.01$, and $\rho_2 = 50$.

Titrations are to be limited to values less than 50 c.c., say $t = t_2 = 47.50$ c.c.

Dilutions of salt solution are to be 10 c.c. salt solution to 1 liter in a liter flask, 10 c.c. of the resultant mixture to be taken in a casserole for titration, or $\rho = 100$.

The concentration of the normal head-water has been previously determined to be 0.0133, as computed from a preliminary test of several salt-solution samples and special dilutions of the sort to be used in the tests.

The ratio of dilution may be first tried at $R = 6\,500$.

Then from Equations (183) and (184), we have

$$c_2 = \frac{2\,906 \times 0.5 \times 0.0133}{(47.50)\left(1 - \frac{5\,000}{6\,500}\right)} = 1.76 \text{ grammes per liter} \quad \ldots\ldots (186)$$

$$c = \frac{6\,500 \times 0.0133}{\frac{6\,500}{5\,000} - 1} = 288.3 \text{ grammes per liter} \quad \ldots\ldots (187)$$

These concentrations, 1.76 grammes per liter for the silver nitrate and 288 grammes per liter for the salt solution, are good practical values, and satisfy the conditions. For a discharge of 1 500 cu. ft. per sec., the rate of discharge for salt solution must be

$$q = \frac{Q}{R} = \frac{1\,500}{6\,500} = 0.231 \text{ cu. ft. per sec} \quad \ldots\ldots (188)$$

which is a trifle more than 6.5 liters per sec.

The salt rate, therefore, is

$$288.3 \times 6.53 = 1.880 \text{ kg. per sec} \quad \ldots\ldots (189)$$

so that, if a test is to last for 15 min. with a margin of 10 min. before the start to secure steady conditions, there will be required

$$1.880 \times 60 \times 25 = 2\,825 \text{ kg. of salt per test} \quad \ldots\ldots (190)$$

which is equivalent to about 3.1 tons.

The total quantity of solution consumed per test would be $6.5 \times 1\,500 = 9\,800$ liters, or about 2 600 gal. This would probably demand a 3 000-gal. tank.

Now, the salt rate and rate of discharge of salt solution are, respectively,

$$\text{Salt rate} = q\,c = \frac{Q\,c}{R} = \frac{Q\,c_1}{\frac{R}{\rho\,\rho_2} - 1} \quad \ldots\ldots (191)$$

and

$$q = \frac{Q}{R} \quad \ldots\ldots (192)$$

It is clear, therefore, that the greater we take R, other things remaining the same, the less the salt requirement and the smaller the salt-solution equipment.

The value of R, however, is limited, to a large extent, by the degree of accuracy required in the test. In order to secure a given degree

of accuracy, for example, it will be necessary to introduce a certain quantity of salt into the head-race to secure a sufficiently large concentration. This will be required, as the titration must not be less than a certain quantity when there is a limit of permissible error.

Perhaps the following is a better procedure than that of the foregoing problem. Suppose the degree of accuracy requires that errors of 1 in 400 shall be easily appreciable. Then, as it is easy to detect 0.1 c.c. on the scale of a 75-c.c. burette, it may be required that sufficient salt, over and above that contained initially in the head-water, shall be introduced to require 40 c.c. of silver nitrate in the titration. The concentration of the salt solution will be 300 grammes per liter, and that of the silver nitrate will be 1.5 grammes per liter. The former is about as strong as it can be taken, and the latter is about as weak as practicable, according to the investigations of Dr. Mellet. The ratio of dilution, $R = R'$, is given by

$$R = 2\,906 \frac{c}{c_2} \frac{S}{t_2} \qquad (193)$$

where t_2 is not the titration of the tail-water, but is that part of it due to the introduction of salt. Thus, $t_2 = 40$ in this case. S is limited by the quantity which may be conveniently taken for evaporation, but should evidently be as large as this limit. A quantity of $\frac{1}{2}$ liter is about as large as most cases will admit. A liter makes a long evaporation and concentrates to a greater extent all the other impurities of the water along with the salt. However, when the method of special dilutions is used, it will be found that liter-evaporations, and even larger, will be possible when the discharge to be determined is very great.

Hence, with $\frac{1}{2}$-liter samples,

$$R = 2\,906 \times \frac{300}{1.5} \times \frac{0.5}{40} = 7\,265.$$

By Equation (184) we find

$$\rho = \frac{R}{\rho_2 \left(1 + R \frac{c_1}{c}\right)} \qquad (194)$$

which, for the foregoing determined value of R, and $\rho_2 = 50$, $V = 0.01$, $c_1 = 0.0133$, becomes

$$\rho = \frac{7\,265}{50\,(1 + 0.322)} = 109.7.$$

Thus we find that

$$1 - \frac{\rho_1 \rho_2}{R} = 1 - \frac{109.7 \times 50}{7\,265} = 0.245$$

and hence, by Equation (183),

$$t_2 = 2\,906 \frac{S}{\left(1 - \frac{\rho_1 \rho_2}{R}\right)} \frac{c_1}{c_s} \quad \ldots\ldots\ldots\ldots\ldots\ldots (195)$$

which, on substitution of values, gives $t_2 = 52.9$, and thus checks the value 12.9 of t_1, given by Equation (173), on addition of 40 to the latter.

The rate of discharge of the salt solution is

$$q = \frac{Q}{R} = \frac{1\,500}{7\,265} = 0.206 \text{ cu. ft. per sec.},$$

which is the equivalent of 5.8 liters per sec., when the discharge to be measured is 1 500 cu. ft. per sec.

The size of the salt-solution tank for a 15-min. test, with 10-min. discharge prior to the start to secure steady flow, is $25 \times 60 \times 5.8 =$ 8 700 liters, and this would require $8\,700 \times 0.300 = 2\,610$ kg. of salt, or about 2.9 tons of salt for each test.

In practice it would probably be better to take $R = 7\,225$, which is the square of 85, and $\rho = 110$. Thus the special dilutions can be quickly made with two 850-c.c. flasks and a 10-c.c. pipette; and the dilution with distilled water can be effected in a flask containing 1 100 c.c., the actual value of ρ having to be computed from the flask and pipette calibrations in any case where precision is to be exacted.

Figs. 7 and 8 will facilitate the determination of the relations between ratio of dilution, size of sample, and titration of the injected salt when the ratio, $c \div c_s$, of the concentration of the salt solution to that of the silver nitrate is 200, which corresponds to concentrations of 300 and 1.5, respectively.

*88.—Care of Samples, and Method for Taking and Treating.**— These operations are among the most important connected with the turbine tests. Consequently, the chemists engaged in this work must take the most scrupulous care to see that samples are properly taken, and not allowed to become contaminated before they are treated in the chemical laboratory.

* Sections 88 and 89 are copied from the original instructions to chemists, with slight modification. They pertain particularly to the method of measuring samples by volume.

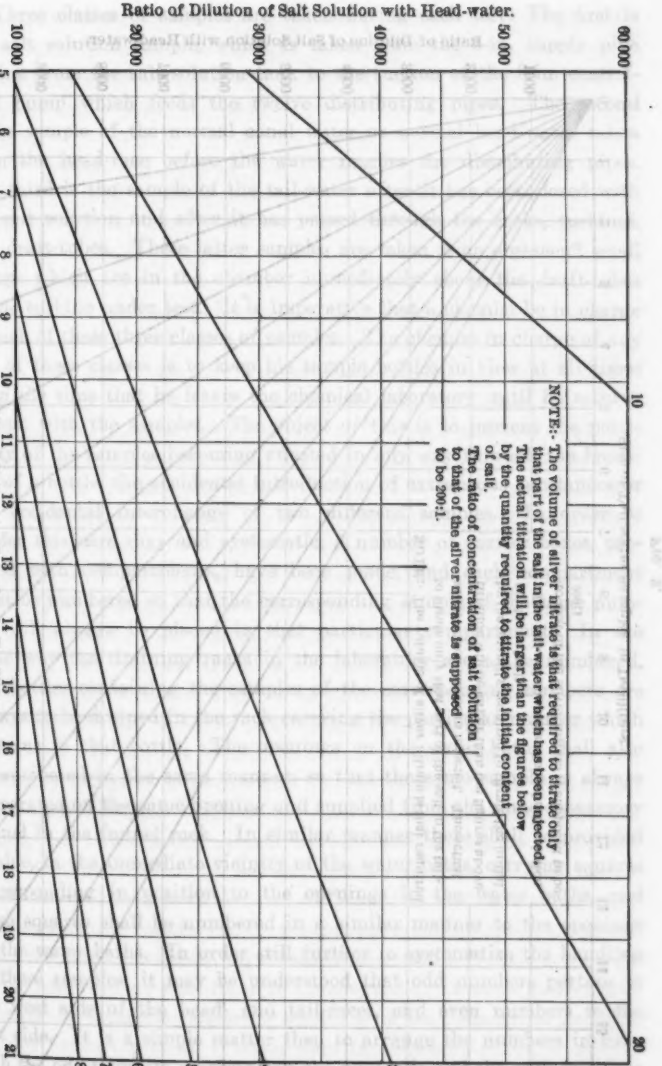

Ratio of Dilution of Salt Solution with Head-water.
10 000
20 000
30 000
40 000
50 000
60 000
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
10
20
80
70
60
50
40
30
NOTE:- The volume of silver nitrate is that required to titrate only that part of the salt in the tail-water which has been injected. The actual titration will be larger than the figures below by the quantity required to titrate the initial content of salt.
The ratio of concentration of salt solution to that of the silver nitrate is supposed to be 200:1.
Size of Tail-water Samples, in Deciliters.
Volume of Silver Nitrate required to Titrate Tail-water Samples, in Cubic Centimeters (when $C+C_s=200$).

FIG. 7.

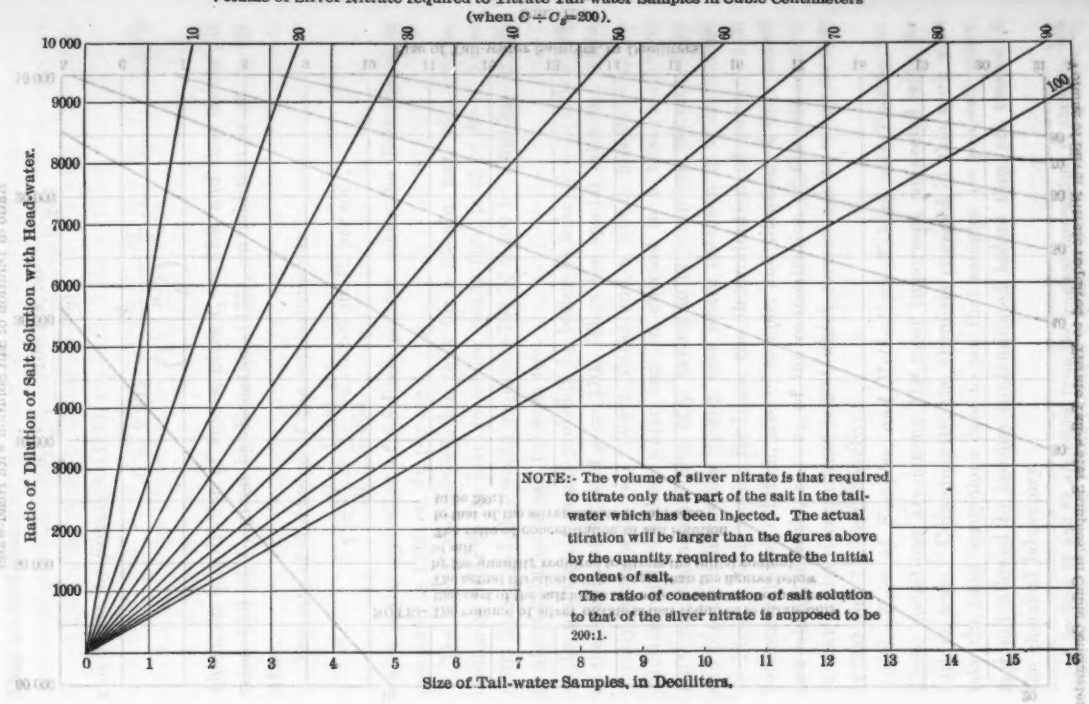

Volume of Silver Nitrate required to Titrate Tail-water Samples in Cubic Centimeters
(when $C \div C_0 = 200$).
10
20
30
40
50
60
70
80
90
100
Ratio of Dilution of Salt Solution with Head-water.
10 000
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Size of Tail-water Samples, in Deciliters.
NOTE:- The volume of silver nitrate is that required to titrate only that part of the salt in the tail-water which has been injected. The actual titration will be larger than the figures above by the quantity required to titrate the initial content of salt.
The ratio of concentration of salt solution to that of the silver nitrate is supposed to be 200:1.

FIG. 8.

Three classes of samples are taken during each test: The first is the salt solution sample, which is taken from the 3-in. supply pipe leading from the salt solution tank to the suction of the 8-in. centrifugal pump which feeds the twelve distributing pipes. The second is the sample of the normal canal water or normal head-water taken from the head-race before the water reaches the distributing pipes. The third is the sample of the tail-water after it has been dosed with the salt solution and after it has passed through the racks, turbines, and draft-tubes. These latter samples are taken from eighteen* small pumps which are in the chamber immediately above the draft-tubes of the turbine under test. It is imperative that a chemist be in charge of each of these three classes of samples. The chemist in charge of any one of these classes is to keep his sample bottles in view at all times from the time that he leaves the chemical laboratory until he returns thereto with the samples. The object of this is to prevent the possibility of the samples becoming affected in any way through the breakage of a bottle, the accidental introduction of extraneous substances or the accidental interchange of two different samples. In order to render this care easy and systematic, a number of carrier boxes, provided with compartments, have been made, and each compartment must be numbered so that the corresponding sample of the same number will always be placed in that particular compartment. In the same way the draining racks in the laboratory are to be numbered, and bottles containing the samples of the corresponding numbers are always to be drained in the rack carrying the particular number which pertains to that bottle. The openings on the water baths shall also be numbered in the same manner, so that the same sample is always evaporated on the same opening and supplied from the same separatory funnel in the funnel rack. In similar manner there shall be provided a table, in the immediate vicinity of the water baths, carrying squares corresponding in position to the openings in the water baths, and these squares shall be numbered in a similar manner to the openings on the water baths. In order still further to systematize the handling of these samples, it may be understood that odd numbers pertain to the west side of the head- and tail-races, and even numbers to the east side. It is a simple matter then to arrange the numbers in rows with the odd numbers representing the west side and the even numbers

* Originally there were only twelve regular tail-water samples.

the east side. The forms for recording observations shall be arranged diagrammatically to correspond to this system. After a complete set of samples has been taken during any test, the bottles are to be tightly corked and carried to the laboratory in charge of the particular chemist whose duty it is to care for the particular class of samples in question.

Normal Head-Water Samples.—The duty of the chemist who is in charge of the normal head-water samples shall be: first, to ascertain the length of the time of the test; second, to ascertain the positions from which normal head-water samples are desired; third, to record the time of beginning and the time of ceasing to take the normal head-water samples. These times shall be recorded by the chemist and turned in with his record of observations during the test. Ordinarily, one normal head-water sample will be taken from each of the four normal head-water sampling pumps during each test. Each sample is to contain approximately 1 gal. The odd-numbered samples shall be taken from the west sampling pipes, and the even-numbered samples from the east sampling pipes; and on every record the position and depth of the intake of the suction pipe on these sampling pumps during the test shall be noted by the chemist in charge of the particular samples in question. At the end of the test these samples are to be carried to the laboratory in the view of, and under the direction of, the chemist in charge. It is an absolute rule in taking these samples that every endeavor be made by all parties concerned to see that the sample fills into the proper sample bottle at as nearly a uniform rate as possible.

Tail-Water Samples.—The chemist in charge of the tail-water samples shall first ascertain the length of time of the test and the time of beginning. He shall then see that the tail-water sampling pumps are started in sufficient time to permit him to gauge the discharges, so that his gallon sample bottles will be nearly, though not quite, filled, during the test. He is then to proceed from the laboratory to the chamber immediately above the draft-tube of the turbine to be tested, where the sampling pumps are placed. The eighteen sample bottles are then to be placed on the movable boards provided for sliding the bottles under the proper sampling pipes. It shall also be the duty of this chemist, as in the case of all other chemists in charge of sampling bottles, to see in advance that each bottle is properly cleaned and prepared for receiving the sample. The greatest care should be taken to see that bottles are thoroughly rinsed with the dosed tail-water and

thoroughly drained before admitting the samples. He shall also keep the samples in his view at all times, so that there will be no possibility of contamination or interchange of samples by accident or otherwise. A few minutes before the beginning of a test, a long ring of the telephone bell will indicate to this sample man that the test is about to begin. A short time later three long rings of the bell will indicate the instant of beginning the test. The test shall begin at the instant when the third long ring begins. One minute after receiving this signal the levers controlling the parts which carry the sampling bottles shall be moved over into such a position that all the bottles will begin to receive their sample at that instant. A few minutes before the end of the test a long ring on the telephone bell will indicate that the test is about to end. Simultaneously at the beginning of the third of the three long rings which follow the preliminary long ring, the test shall be at an end, and 1 min. thereafter the levers controlling the boards carrying the tail-water samples shall be moved in such a manner that the bottles will cease to receive any water from the sampling pipes at that instant. The chemist in charge of the tail-water samples shall then proceed as quickly as possible to have his samples carefully carried to the chemical laboratory and properly cared for thereafter.

Salt-Solution Samples.—These samples shall be taken from the 3-in. pipe leading from the salt-solution tank to the suction of the 8-in. centrifugal pump at a point as near as practicable to the suction pipe. There shall be five of these 1-qt. samples, marked A, B, C, D, and E. It shall be the duty of the chemist in charge of these samples: first, to ascertain the time of duration of the test; second, to divide that time into five equal parts; and third, to see that the samples are taken in sequence, beginning with A and ending with E, each one being held under the sample cock for one-fifth of the duration of the test. It shall also be his duty to see that the sampling cock gives a uniform discharge during the taking of any one sample, and that this rate of discharge is such as nearly to fill the bottle during the particular time devoted to the particular sample in question. All the other requirements for cleanliness and nicety in the work connected with sampling shall be observed by all the chemists, in order to secure the desired degree of accuracy for the turbine tests. In particular, all sample bottles, after having been previously cleaned, shall be rinsed with a

portion of the sample water before being filled with another portion which is to constitute the sample proper.

Reagents.—There are three principal reagents connected with the turbine tests. The first is the commercial salt, which has been purchased in large quantity, for the purpose of making the salt solution. This salt contains more or less sand, also pieces of paper, chips of wood, and other extraneous substances, which are largely screened out during the process of charging the salt-solution tank. About 12 000 lb. are to be dissolved in the 5 000-gal. tank provided for the purpose. The resulting solution is then injected into the head-water through the twelve distributing pipes. The object of these distributing pipes is to assist in getting a good mixture of the salt solution with the feed-water for the turbine.

Silver Nitrate.—The second reagent used in this work is the centi-normal, or approximately centi-normal, solution of silver nitrate. Although it is not necessary that this solution should be standardized with the highest degree of accuracy, it is desirable that some degree of precision be required in determining the weight of silver nitrate dissolved per liter of distilled water. This is for the purpose of securing uniformity of strength from day to day. The general procedure for making the silver nitrate solution shall be as follows: About 14.5 grammes of silver nitrate shall be dissolved in 1 liter of distilled water. In weighing this quantity of silver nitrate for this purpose, the margin of error allowed in the weight may be taken at 0.05 gramme. After this solution is made to the extent of 1 liter in a dark-colored bottle, it shall be carefully preserved in a covered box where it will be entirely protected from the light except at short intervals when the solution is to be taken from the bottle. The solution required in the titration is to be made up from this solution at various times when needed, by diluting 100 c.c. to 1 liter with distilled water. In case larger quantities are needed, correspondingly larger quantities may be diluted.

It is an absolute rule, in reducing tests in the laboratory, that the same silver nitrate solution be used during any one particular test, as this rule is imperative in order to eliminate the errors of titration due to the excess of silver nitrate required to give a definite end reaction and otherwise to secure precision.

Chromate of Potassium.—Chromate of potassium shall be used as an indicator. This solution shall be made by dissolving 50 grammes in

1 liter of water, or, if larger or smaller quantities are desired, in corresponding proportions.

Quality of Reagents.—In the case of all the foregoing reagents, except the salt, chemically pure chemicals shall be used.

Titrations.—All the samples are generally taken and treated in such a way that the titration will require about 50 c.c. of silver nitrate. It will be necessary, therefore, to use 100-c.c. burettes, as, in many cases, the titration may exceed 50 c.c. Further than this, the laboratory procedure reduces each sample, either by dilution or evaporation, to about 10 c.c. in a casserole, so that the titrations of all samples are nearly identical in character, as the samples contain nearly the same quantity of chlorides at approximately the same concentration. Wherever possible, the method of balanced evaporations, Section 45, shall be used in concentrating by evaporation.

In order to secure uniformity of treatment, several rules must be strictly observed by the chemists when titrating samples.

Procedure.—

1.—The burette shall be filled to the zero of the scale, as nearly as practicable without loss of time, say to 0.4 or 0.3 c.c. Care must be taken to scrape off all surplus solution from the discharge orifice of the burette with a clean glass rod and to see that there is no surplus solution clinging to the top of the burette tube on the inside where it would be likely to fall into the solution below, thus causing an error in the estimate of the quantity of nitrate used in the ensuing titration.

2.—Take a reading of the burette.

3.—Record this reading on the form for the purpose, also noting the number of the burette beside this reading.

4.—Place the casserole containing the samples to be titrated under the burette.

5.—Introduce one drop of indicator into the sample by using a dropping bottle.

6.—One minute, approximately, after the first reading of the burette, take a second reading and record it immediately under the first. In general, these readings will be found to agree. If not, wait another minute, take a third reading, and record it immediately under the second. The third reading should agree with the second. If not, continue to take and record readings one minute apart until two consecutive readings, taken in this manner, agree.

7.—The stop-cock of the burette is now to be adjusted so that about two drops per second of silver nitrate will fall into the sample being titrated. The burette must be adjusted in the support so that the drops will not fall far enough to cause any splashing of the solutions, thus preventing losses over the side of the casserole.

8.—While the silver nitrate is thus being discharged into the sample, the observer has time to rinse a glass stirring rod with distilled water and to stir the sample with it in order to secure a thorough mixture, meantime watching the level of the silver nitrate solution in the burette and keeping the stop-cock adjusted so as to maintain the rate of discharge at about two drops per second.

9.—For each 10 c.c. of silver nitrate admitted to the sample in titration, one drop of chromate of potassium indicator is to be added to the sample as the level of the silver nitrate solution falls past the corresponding graduation mark on the burette. The object of this is to keep the quantity of indicator present in the sample proportional to the quantity of silver nitrate consumed, and also proportional to the volume of solution in the casserole. The color of the sample is also thus preserved as nearly uniform during the titration as possible.

10.—As soon as the titration nears the end point, the orange tint which appears on the admission of each drop of nitrate will tend to persist a longer time as the solution is stirred continuously with the glass rod. A little experience with samples of a particular class will enable the observer to tell when the titration is within about 2 c.c. of the end.

11.—When the titration is thus brought within 1 or 2 c.c. of the end, the observer must readjust the stop-cock of the burette so as to retard the rate of the discharge of silver nitrate to about one drop in 2 or 3 sec., stirring the solution thereafter continuously and scrutinizing the changing tints of the solution closely. Finally, a drop of silver nitrate will fall into the casserole which causes the solution to take a faint, though perfectly definite, orange tint. When this occurs the observer must be prepared to close the stop-cock instantly so as to stop the discharge from the burette. On continuing the stirring for 30 or 40 sec., the orange tint may gradually fade out, giving place to the former lemon-yellow tint. If this is the case, add one more drop of silver nitrate from the burette, and continue the stirring. By pursuing this course, a drop from the burette will be finally admitted into the casserole which will bring the solution to a very faint but definite

and permanent orange tint. This is the end point sought. Care must be taken before reaching this stage to see that there is no surplus drop of silver nitrate clinging to the bottom of the burette after reaching the end point, otherwise the burette reading will be erroneous, and an error will have been committed. The best safeguard at this stage is to remove each drop as it forms at the tip of the burette with the end of the glass stirring rod and introduce it in this way into the solution, closing the stop-cock each time before removing the drop.

12.—Read the burette scale and record the reading on the form for the purpose. A minute later check the reading and indicate the check by a check mark opposite the last reading.

If the solution has not been drawn from the burette faster than permitted by these rules, it will rarely be found that a third reading at the end of the titration is necessary.

13.—The observer is now to compute the number of cubic centimeters of silver nitrate consumed in the titration. After checking his calculation carefully, he is then to affix his initials opposite the result, and proceed to another titration, the sample for which shall be selected by the rules for group titrations, Section 48.

Preparation of Salt-Solution Samples.—

The operations in the laboratory should be arranged so that the various solutions will come into contact with nothing but glass or porcelain. For this reason the writer is not an advocate of the use of the "policeman" with a rubber or cork tip.

Special pipettes and glassware should be reserved exclusively for the strong salt solution. This will save much time in rinsing, and insure accuracy in the corresponding parts of the work.

1.—Shake the bottle containing the sample of salt solution to be treated.

2.—Take a clean 10-c.c. pipette, rinse it with a portion of this sample, allowing this portion to be discharged into a sink or waste receptacle.

3.—Fill the pipette again from the sample and withdraw very slowly from the solution so as to drain the outside of the pipette thoroughly.

4.—Let the sample in the pipette sink slowly to the graduation mark in the usual manner and, if necessary, scrape off the surplus drop at the tip with a clean dry glass rod.

5.—Also examine the tip and outside of the pipette to see if there is any surplus solution which can in any manner become incorporated

by accident with the solution to be delivered by the pipette. If so, it must be removed, being careful not to contaminate the contents of the pipette.

6.—Discharge the pipette into a liter flask, which has been previously cleaned and rinsed with distilled water. The neck of this flask must be free from drops of distilled water when the salt solution is introduced. The manner of discharging will depend on the manner of calibrating the pipette by the chemists.

7.—Fill the flask to the graduation mark with distilled water, stop the bottle, invert a few times, and add a few more drops, if necessary, to correct the shrinkage. Make a perfect mixture by stopping the bottle and inverting it from thirty to forty times, giving it a shake each time and allowing the air to pass from top to bottom and bottom to top after each inversion.

8.—Take the temperature of both the salt solution and the distilled water immediately after the sample, or samples, have been diluted as just described.

9.—Record these temperatures on the forms provided for the purpose, and observe whether the level of the solution in the flask when held vertical is still at the graduation mark.

10.—A 10-c.c. pipette full of the dilution just described delivers into a casserole a sample of dilute salt solution ready for titration. Chemists must observe the rules already given for the discharge of pipettes.

11.—Take and record the temperature of the dilution in the liter flask immediately after charging all the casseroles which are to contain samples for titration from this flask.

12.—Empty all flasks and other receptacles containing solutions for which there is no further use.

Normal Head-Water Samples.—These samples are of the normal canal water taken in the head-race before the water receives the salt solution from the distributing pipes. Four 1-gal. sample bottles are nearly filled with this water. Dilutions of salt solution with portions of these samples are prepared for each test. As a general rule, only one rate of dilution will be used in reducing any given test.

Special Dilutions.—Several different rates of dilution, however, may be used in preparing the normal head-water samples for titration.

Three examples follow. Observe carefully the general rule for making accurate dilutions, Section 89.

Dilution A.—Take five 10-c.c. pipettes of salt solution and dilute with 3 liters of normal head-water, thus making approximately 3 050 c.c. of stock solution in a gallon bottle. Shake thoroughly.

A_{123}.—One of these 3 liters for the dilution must come from each of the bottles containing normal head-water samples 1, 2, and 3. Label the stock solution thus prepared A_{123}.

A_{234}.—In precisely similar manner prepare another stock solution, taking 1 liter each from normal head-water samples 2, 3, and 4. Label this stock solution A_{234}.

Dilution B.—Take four 10-c.c. pipettes of salt solution and dilute with $2\frac{1}{2}$ liters of normal head-water, thus making approximately 2 540 c.c. of stock solution in a gallon bottle. Shake thoroughly.

B_{12}.—One of these $2\frac{1}{2}$ liters for the dilution must come from normal head-water sample bottle 1 and the remaining $1\frac{1}{2}$ liters from bottle 2. Label this stock solution B_{12}.

B_{34}.—In similar manner prepare another stock solution, taking 1 liter from normal head-water sample 4 and the remaining $1\frac{1}{2}$ liters from normal head-water sample 3. Label this stock solution B_{34}.

Dilution C.—Take three 10-c.c. pipettes of salt solution and dilute with 2 liters of normal head-water, thus making approximately 2 030 c.c. of stock solution in a gallon bottle. Shake thoroughly.

C_{12}.—One of these liters for the dilution must come from each of the bottles containing normal head-water samples 1 and 2. Label this stock solution C_{12}.

C_{34}.—In similar manner prepare another stock solution, taking 1 liter from each of the bottles containing normal head-water samples 3 and 4. Label this stock solution C_{34}.

89.—To Make Accurate Dilutions with the Chemical Equipment in the Laboratory.—Let it be required to dilute n pipettes of salt solution with m liter flasks of solution which will be called water, the whole to be mixed in a larger receptacle. Use only flasks which contain a given quantity of water, nominally 1 liter. Divide as nearly as possible the n pipettes of salt solution among the m flasks, discharging them into the flasks according to the rules for pipettes, and preparing the flasks according to the rules for flasks.

Fill the flasks to their graduation marks with water and shake thoroughly, according to the directions already given. Let the flasks rest on a table for a few minutes and then, if necessary, add a drop or two more of water to restore the level of the meniscus to the graduation mark.

Empty all the flasks into the receptacle which has already been cleaned, rinsed with distilled water, and drained, as previously directed for flasks. Shake thoroughly, empty the contents, and repeat the entire procedure, except that the final mixture in the receptacle is to be retained as the desired mixture.

The total contents of the flasks divided by the total discharge of the pipettes is the ratio of dilution.

In case it is desired to make a dilution up to a given ratio of mixture, the rule is correspondingly the same as the foregoing except that the n pipettes are discharged into the receptacle instead of the flasks, after which the m flasks of water are poured in. The flasks are finally rinsed into the receptacle with the final mixture therefrom. The resulting ratio of mixture is the total contents of the m flasks divided by the total discharge of the pipette, or pipettes, as the case may be. When n is not a multiple of m it will be on the side of precision to dilute for ratio of mixture.

Further Treatment of Special Dilutions.—In every test, therefore, there will be two stock bottles containing the special dilutions or stock solutions. These special dilutions are to be further diluted by discharging a 10-c.c. pipette of the special dilution into a liter flask which has been previously cleaned and rinsed with normal head-water. The flask is then filled with normal head-water in the usual manner and the stopper tightly adjusted in the neck. The flask must then be alternately inverted and righted forty times, allowing the air bubble to rise to the highest point within the flask on each half turn. When satisfied that the mixture is uniform, the sample is to be divided into two half-liter samples by pouring it into two half-liter flasks which have been previously rinsed with distilled water, the liter flask containing the original sample being finally rinsed with distilled water, which may be divided between the two half-liter flasks. Each of these half-liter samples is then to be poured into the particular separatory funnel which has been reserved for it. The flask is then to be rinsed

with a small portion of distilled water which is finally poured into the separatory funnel.*

Prepare ten such half-liter samples altogether, taking four from one of the stock bottles of special dilution and the remaining six from the other stock bottle. Mark each casserole, with a label attached to the handle, with the symbol which indicates the particular stock bottle out of which the sample to be evaported in the casserole was taken.

These samples are next to be evaporated over the water-bath, the attendant maintaining a volume of about 40 c.c. in each casserole during the evaporation. This is effected by drawing from the separatory funnel over each casserole at proper intervals of time sufficient solution to restore the level of the liquid in the casserole to the proper elevation.

When all the sample which will discharge from any one funnel into its casserole has been drawn off in this manner, the attendant must rinse down the sides of the funnel with a small quantity of distilled water which is also admitted to the casserole at the proper time.

Great care must be taken in all the operations connected with these evaporations to see that no part of any sample is lost, and that all the salt of any sample is finally collected in the proper casserole, also to see that the level of the solution in every casserole is maintained as nearly fixed as reasonable care can insure.

When all the salt from any sample is collected in this manner, there should be about 40 c.c. of solution in the casserole. The evaporation is then to be continued until the volume of the liquid in the casserole is reduced to about 10 c.c.

This can be accurately estimated by the attendant by comparison with the volume of water, in another casserole of the same size and shape, which results from the delivery of a 10-c.c. pipette.

The casserole containing this final concentration of salt is then to be placed on the square reserved for this purpose on the sample table.

This sample is then ready for titration.

Tail-Water Samples.—There are eighteen of these samples, taken in gallon bottles from eighteen different locations in the tail-race. The numbers are not consecutive, being, in particular, 21 to 32, both

* This procedure was adopted so as to make effective use of the particular glassware in the laboratory.

inclusive, 41, 42, 43, 51, 55, and 56. This irregularity in numbering resulted from a series of changes which were made in the locations of the tail-race sampling pipes.

Evaporation of Tail-Water Samples.—One-half liter is measured from each of the eighteen samples into the corresponding separatory funnel, the half-liter flask in each case being rinsed with a small portion of the sample before being filled with it and with a small portion of distilled water after being emptied, this portion of distilled water finally being poured into the separatory funnel to prevent any loss of salt.

The evaporation is then to be conducted in precisely the same manner as that given for the evaporation of the special dilutions.

Modified Procedure for Tail-Water Samples.—It is desirable in some cases to reduce the number of evaporations of tail-water samples. In order to make the process clear, it will aid to observe that the number and relative location of each sample in the tail-race is indicated by the following diagrammatic tabulation:

UPPER DRAFT-TUBE (LOOKING UP STREAM).

WEST.		EAST.	
43	29	55	
23		32	56
51	31	41	

LOWER DRAFT-TUBE (LOOKING UP STREAM).

WEST.		EAST.	
25		28	22
21	27	26	
42		30	24

The samples, therefore, may be classed according to the particular half of the draft-tube to which they belong, thus:

UPPER DRAFT-TUBE.	WEST HALF	43	29
		23	
		51	31
	EAST HALF	55	
		32	56
		41	

Lower Draft-Tube.	West Half	25	
		21	27
		42	
	East Half	28	22
		26	
		30	24

The samples represent approximately equal areas of cross-section of the draft-tube. The samples of any half of a draft-tube, therefore, will be combined by mixing equal parts, say ½ liter of each, in a gallon bottle which has been previously cleaned and then rinsed with a mixture of approximately equal portions of the corresponding samples.

Thus, for the west half of the upper draft-tube, mix ½ liter of each of Nos. 43, 51, 23, 29, and 31. For the west half of the lower draft-tube, mix ½ liter of each of Nos. 21, 25, 42, and 27.

Shake the resulting stock solutions, consisting of 2 or 2½ liters in gallon bottles, thoroughly, and label the bottles according to the following scheme.

Upper Draft-Tube, West Half..... U. W.
Upper Draft-Tube, East Half..... U. E.
Lower Draft-Tube, West Half..... L. W.
Lower Draft-Tube, East Half..... L. E.

Measure accurately from each stock solution two ½-liter samples which are to be placed in the separatory funnels reserved for the particular samples in question.

In all cases the volumetric flasks must be cleaned and rinsed with a portion of the stock solution from which the sample is to be taken before being filled with the sample. Moreover, the flask from which any sample has been discharged into its separatory funnel must be finally rinsed with a small portion of distilled water, which portion must then be discharged into the separatory funnel, so as to insure the capture of all the salt of the sample.

Each casserole must finally be labeled with the symbol which pertains to the particular stock solution which is to be evaporated in it.

In all cases the same separatory funnel and location on the water-bath should be reserved for the same sample in different tests.

PART III.

THE TESTING PLANT.

SUMMARY.

90.—In Part III the general plan for the salt-solution tank and distributing system is described.

The current-meter rack and platform, which also carries the salt-solution distributing pipes, is described and illustrated. This structure with its equipment can be moved bodily by the crane from one unit to another.

The salt-dissolving box and platform is illustrated. By this invention 7 tons of commercial salt can be put into solution in a 5 000-gal. tank within 1 or 2 hours, according to circumstances. Its operation is continuous. The solution is very nearly saturated.

The sprinkler system is described and illustrated.

The large iron calibrator, or pipette, is described.

The tail-race sampling pumps are illustrated and the method of operating them explained.

The chemical laboratory and equipment and the direct-current metering instruments are illustrated.

As the method of measuring the discharge of hydraulic turbines by a chemical introduced into the feed-water is a relatively new branch of engineering, especially when applied to hydro-electric units of large capacity, a somewhat detailed description of the chemical testing plant, as it was installed for the tests considered herein, will be necessary, in order to make clear the exact nature of the operation of testing as it was conducted by the writer. As a matter of fact, the methods were entirely novel in nearly all respects.

The magnitude of the chemical operations conducted may be understood better when it is explained that each test required from 4 to 6 tons of commercial salt, which was used as a chemical reagent. The problem of getting this quantity of salt into a solution very near to the saturation point alone offered considerable apparent difficulty at the outset. Numerous preliminary tests were made to ascertain the probable length of time which would be required to make such a solution in a 5 000-gal. tank, with the apparent result that it would require several hours and great care. It was apparently very difficult to make

a saturated solution of 1 liter capacity in a short period of time. On careful reflection, however, it was decided to make what the writer terms a dissolving box, and this invention proved to be very efficient in producing salt solutions nearly up to saturation. Moreover, it was found by actual test that 7 tons of salt could be dissolved and the resulting solution pumped into a 5 000-gal. tank in less than 1 hour.

In the case of tests which had been conducted in Europe, on turbines discharging a very small quantity of water, it appears to have been the custom to filter the salt solution before admitting it to the solution tank. This appeared to the writer to present too many chances for delay, and, therefore, it was decided to try out a method which would not demand the additional time necessary for filtration.

91.—General Plan for Salt-Solution Tanks and Distributing System.—Fig. 9 is the writer's original diagrammatic sketch for the salt-solution tanks and distributing system. Very little change was made in this general plan when laying out the details of the actual system as it was installed. On the left is the dissolving box, which is substantially of the same dimensions as the one actually used. Instead of a 3-in. pipe, however, leading to the salt-solution screens, as shown in the sketch, a trough about 8 by 10 in. in cross-section led from the top of the dissolving box on a steep slope to a point where its discharge was emptied into a hood fitting over the end of the trough, hanging vertically therefrom and provided with screens of very fine mesh in the bottom. This hood could be taken from the trough at any time for the purpose of emptying the accumulations of rubbish. The latter consisted principally of chips of wood and other extraneous matter which was included as impurities of the low commercial grade of salt used for the tests. At several points in the trough there were screens of somewhat coarser mesh than those at the bottom of the hood. These were slipped into slots in the sides of the trough and served to remove the coarser impurities without allowing them to fall into the hood. The salt solution, after passing through the screen at the bottom of the hood, fell into a 5 000-gal. tank. It was originally intended to have a 6 000-gal. tank, but the one supplied contained only 5 000 gal. and appeared to fulfill all necessary requirements. The hood into which the salt solution fell from the trough was approximately 12 in. wide, 15 in. long, and 18 in. high; the screen, therefore,

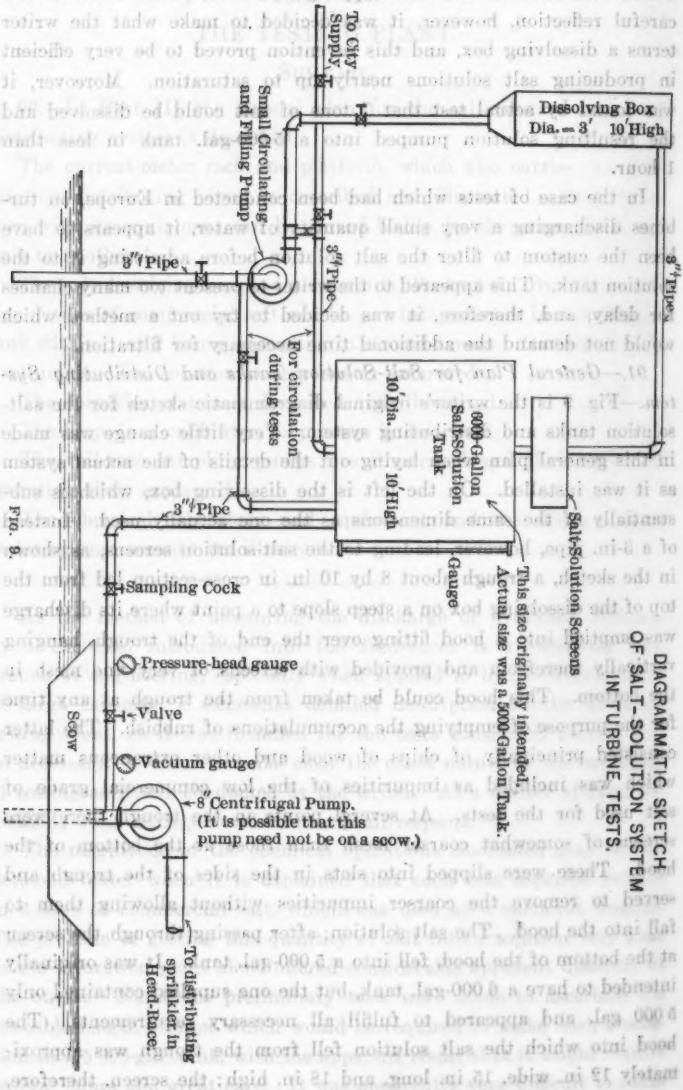


FIG. 9. DIAGRAMMATIC SKETCH OF SALT-SOLUTION SYSTEM IN TURBINE TESTS.

offered an effective area of about 12 by 15 in. This screen had forty meshes to the inch.

At the side of the salt-solution tank there was a 10-ft. gauge-board, having a vertical slot along the axis of the board sufficiently deep and wide to admit a glass tube of 1 in. inside diameter, the bottom of which was connected to the bottom of the tank by a short length of 1-in. rubber tubing. There were also two valves, one being at the bottom of the glass tube for the purpose of emptying the gauge glass in order to admit fresh solution from the tank. The object of this operation was to have the gauge glass full of solution of the same density and at the same temperature as that contained by the large tank. The other valve was at the end of the 1-in. rubber tubing, and served to cut off the flow from the tank when it was desired to empty the gauge glass. It is important to have the valve which empties the gauge glass immediately at the bottom of that glass, rather than next to the valve beside the tank. The object of this is to empty the gauge glass without completely emptying the hose, otherwise, when it is attempted to fill the gauge glass again, air is likely to be entrained in the solution and thus cause some difficulty in securing a perfect meniscus in the gauge glass.

A small 3-in. centrifugal pump, driven by a small motor, was used to circulate the water. It was connected, in about the manner shown diagrammatically in the sketch, to the dissolving box and salt-solution tank. Instead, however, of its suction dipping down into the head-race, as shown in the sketch, it was connected directly to the city water supply system. In using the city water supply, the pump tender must take great care that the full city pressure is not allowed to act on the pump casing, as it is likely to burst.

It will be seen from the arrangement of the 3-in. piping and valves that the circulating pump serves a double purpose. It is capable of circulating the water up through the dissolving box, over through the 8 by 10-in. trough and into and through the 5 000-gal. tank; or, it may simply circulate the solution in the tank by drawing it off through the suction and discharging it back into the bottom, the salt-dissolving box under these circumstances being cut off by a valve in the 3-in. pipe. The object of this latter operation is to secure a perfect mixture in the salt-solution tank at a uniform temperature; also to dissolve

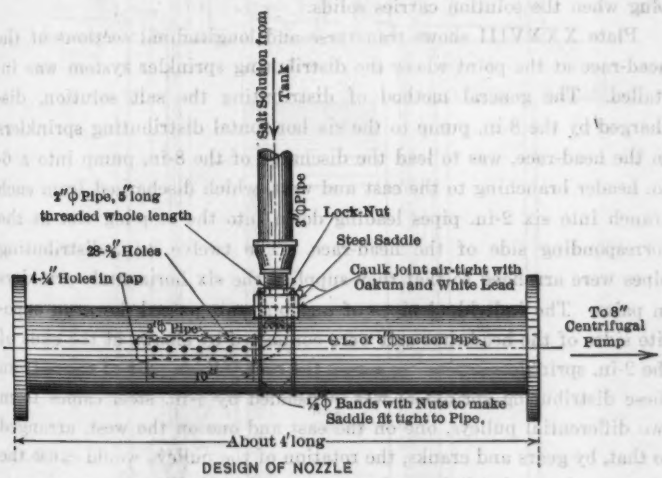
any salt which may find its way into the 5 000-gal. tank and settle to the bottom.

The operation of charging the tank with salt solution consists in, first, emptying and cleaning the large tank. This may be done through an opening in the bottom which is provided with a large nipple and gate-valve. The small circulating pump is then started, the valve leading to the dissolving box being open and the water being taken from the city supply, the valves in the 3-in. pipes leading to the large tank, of course, being closed. As soon as the water begins to flow into the dissolving box, salt is shoveled in at the top. The salt settles down through the water which is rising in the dissolving box and flowing over into the large tank through the 8 by 10-in. trough. The rate of water supply and the rate at which salt is shoveled into the top of the dissolving box must be properly adjusted by trial in order to secure the maximum efficiency. It was found that the capacity of the circulating pump was just about sufficient, when somewhat throttled down, to dissolve salt at the rate at which two men could shovel it in at the top. One or two attendants are required to keep the screens free from débris as the water flows into the solution tank. It was not found necessary to remove the hood during the operation of charging the tank, and this, indeed, would not be desirable at any time during the operation, as it would be likely to allow the admission of débris into the solution tank with the consequent danger of plugging the distribution system, especially at the point where the water is admitted into the suction of the 8-in. centrifugal pump which supplies the distributing sprinklers.

There was always a considerable percentage of fine sand carried by the salt, and this passed through all the screens and fell into the large tank, after which it settled on the bottom to a depth of $\frac{1}{8}$ to $\frac{1}{4}$ in. for each charge. The tank was not always cleaned after each filling, as this layer of sand caused no inconvenience. The sand was very fine and compact, and was not likely to be dislodged by currents of salt solution in the tank. The time required to charge the tank varied from somewhat less than 1 hour to 2 hours or more, according to circumstances.

A 3-in. pipe led from the bottom of the salt-solution tank to the suction of the 8-in. centrifugal pump, shown on a scow in Fig. 9. This scow was not used at any time in the tests. Originally, the object in

having the scow was to maintain a constant suction head on the large centrifugal pump, but it was not found necessary to have a uniform supply of water from this pump and consequently no scow was necessary. Had the scow been used, it would have been necessary to have flexible joints in the pipe lines leading to the centrifugal pump. As a matter of fact, the pump was on the deck of the power-house platform immediately over one of the piers of the head-race. In this position the suction lift varied from 17 to 23 ft., but the pump seemed to work very satisfactorily under any of these conditions of lift. There



DESIGN OF NOZZLE
FOR INJECTING SALT SOLUTION
INTO SUCTION OF 8-INCH CENTRIFUGAL PUMP.

FIG. 10.

was no screen over the intake of the 3-in. pipe leading to the 8-in. pump. The writer is inclined to believe that it would have been wiser to have a screen over the intake of the 3-in. pipe, although there was no difficulty in this respect during any of the tests.

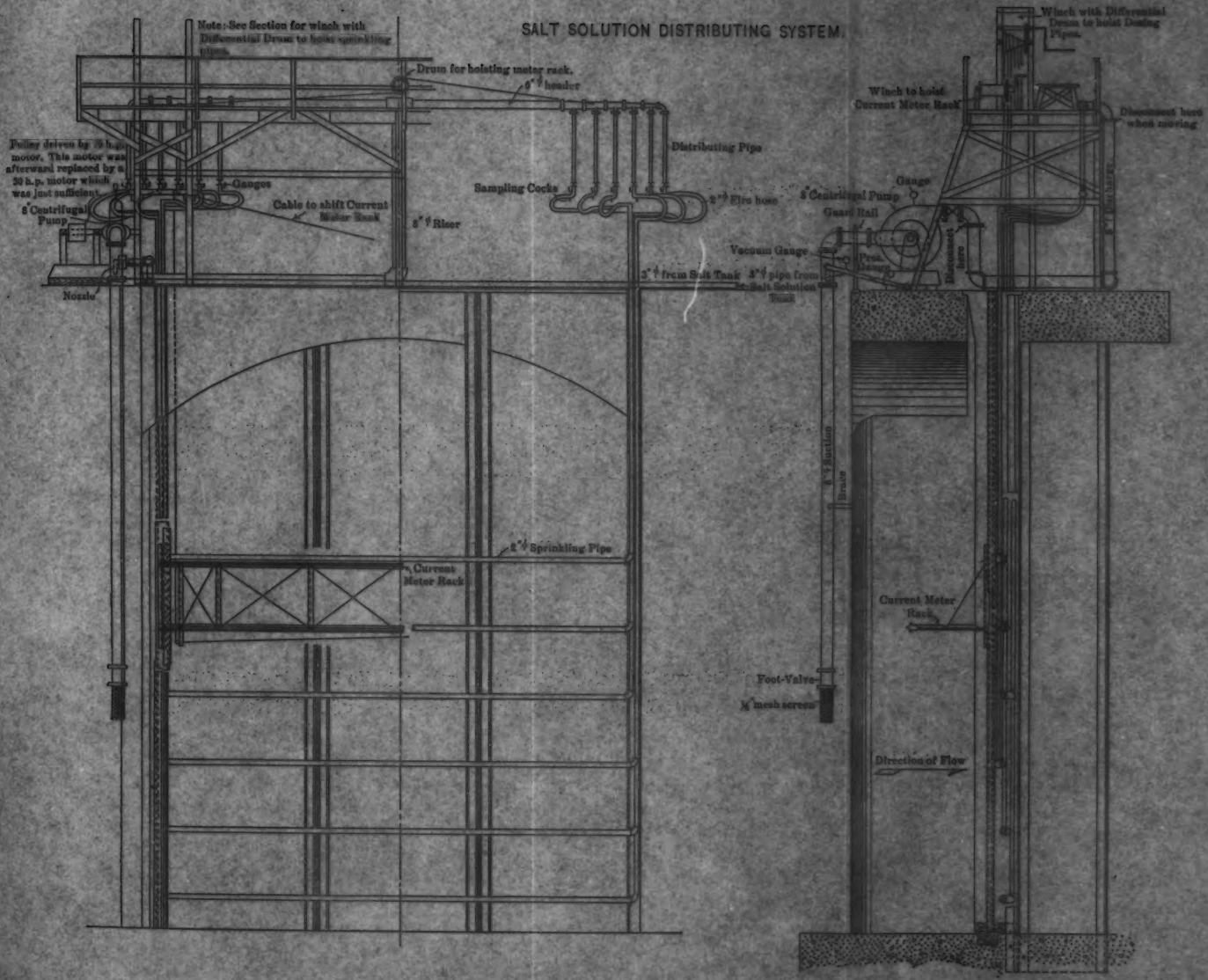
Fig. 10 shows diagrammatically the method of introducing the salt solution into the suction of the 8-in. centrifugal pump. The sprinkler attached to the elbow within the 8-in. suction points against the flow of the water on its way from the canal water level to the pump. This sprinkler was 10 in. long and had twenty-eight ⅜-in. holes through its convex surfaces, and four ¼-in. holes in the cap.

It may not have been necessary to provide the end of the 3-in. salt-solution pipe with such a sprinkler, but the writer is of the opinion that this aids materially in securing a perfect mixture of salt solution with the feed-water for the centrifugal pump. There was little time for experimentation in this respect, so that there was no evidence whatever bearing on the design of such a sprinkler. It would appear to the writer that the holes for this sprinkler should have been smaller, so as to project jets of solution as far as possible into the body of the suction water feeding the pump. If the holes are too small they will plug when the solution carries solids.

Plate XXXVIII shows transverse and longitudinal sections of the head-race at the point where the distributing sprinkler system was installed. The general method of distributing the salt solution, discharged by the 8-in. pump to the six horizontal distributing sprinklers in the head-race, was to lead the discharge of the 8-in. pump into a 6-in. header branching to the east and west, which discharged from each branch into six 2-in. pipes leading down into the stop-log slot at the corresponding side of the head-race. The twelve 2-in. distributing pipes were arranged so that they supplied the six horizontal sprinklers in pairs. The individual pipes of any one pair passed down on opposite sides of the head-race and were connected by elbows at the ends of the 2-in. sprinklers extending across the race. Each pair of risers from these distributing sprinklers was suspended by ¼-in. steel cables from two differential pulleys, one on the east and one on the west, arranged so that, by gears and cranks, the rotation of the pulleys would cause the pipes to rise and fall in such a manner that they were always equidistant, thereby dividing the cross-section of the head-race into six equal horizontal strips. The object of this was to keep the salt solution as uniformly distributed over the cross-section of the head-race as possible, and to permit of adjusting the levels of the pipes to the level of the surface of the water in the canal, which was likely to vary 6 or 8 ft. from time to time.

The necessity for this adjustment demanded flexible connections between the distributing pipes leading from the 6-in. header and the risers leading up from the sprinklers. This was accomplished by using sections of 2-in. hose of sufficient length to admit of the necessary motions. See Figs. 22 and 23.

SALT SOLUTION DISTRIBUTING SYSTEM.



The 6-in. header which has just been briefly described was carried across the top of a wooden platform resting on a frame, which is shown by Fig. 11. This will be better understood by examining Figs. 22 and 23, which are explained in detail in Section 97.

92.—Current-Meter Rack and Platform.—The platform, Fig. 11, which has just been mentioned, was constructed primarily for the purpose of suspending the current meters, which were carried on a steel frame or rack. This meter rack, Fig. 12, was constructed in two sections or leaves which were capable of folding so as to admit of the whole being dropped through the head-gate opening and into the stop-log slots, where it rested against two steel guides composed of two channels. These channels were provided at the bottom with a pair of legs, or spreaders, so that the channels could be dropped down the stop-log slots in a vertical position in such a manner that when the spreader came in contact with the bottom the legs would spread apart and hold the channel rigidly in its proper position at the bottom. The channels were then made fast at the top, so as to be practically in a vertical position, thereby acting as guides for the current-meter rack at each side of the head-race. The object of this arrangement was to admit of placing and removing the guides without the necessity of shutting down the unit or emptying the head-race. These channels seemed to work very satisfactorily, and could be placed and removed in a few minutes. The current-meter rack was suspended at each end by a ¼-in. steel cable which led vertically up to a pulley at the corresponding end of the current-meter platform; these two cables then passed around the drum of a small winch, by which the rack could be raised or lowered at pleasure.

In order to lower the rack into the head-race, its two leaves were closed and the whole was dropped through the head-gate opening into the stop-log slot until it was low enough to admit of opening the horizontal leaf into position. The current meters had been previously attached to the rack, so that they were then ready for operation. The rack with the current meters could thus be lowered to the bottom of the head-race or to any intermediate position desired. The horizontal leaf of the rack operated on vertical pivots, so that it could be shifted toward one side or the other of the race, as indicated by the dotted lines in Fig. 12, only one-half of the meter rack being shown. The rack was equipped with five pieces of angle iron, about 1 ft. 7½ in.

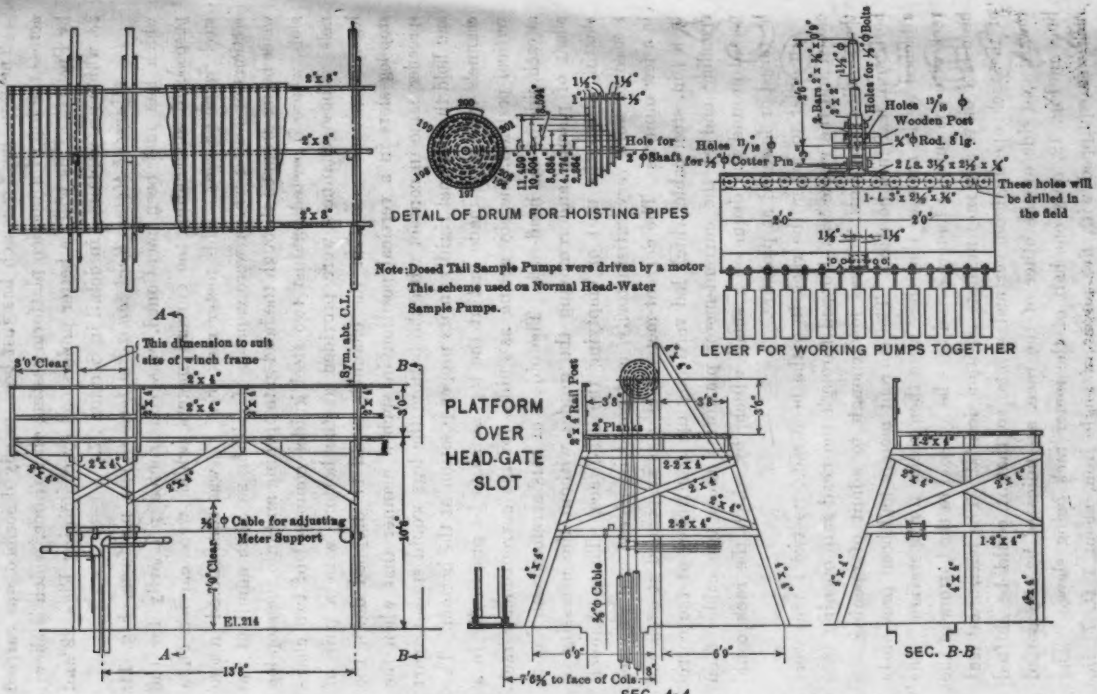

DETAIL OF DRUM FOR HOISTING PIPES
Hole for 3" ϕ Shaft
Holes 11/16 ϕ for ½" ϕ Cotter Pin
Holes for ½" ϕ Bolts
Holes 11/16 ϕ Wooden Post
½" ϕ Rod. 5' lg.
These holes will be drilled in the field
LEVER FOR WORKING PUMPS TOGETHER
Note: Dosed Tail Sample Pumps were driven by a motor This scheme used on Normal Head-Water Sample Pumps.
This dimension to suit size of 7-inch frame
3'0" Clear
8'm. abt. C.L.
¾" ϕ Cable for adjusting Meter Support
El. 214
13'6"
PLATFORM OVER HEAD-GATE SLOT
2" Planks
6'9"
7'6⅜" to face of Cols.
SEC. A-A
SEC. B-B

FIG. 11.

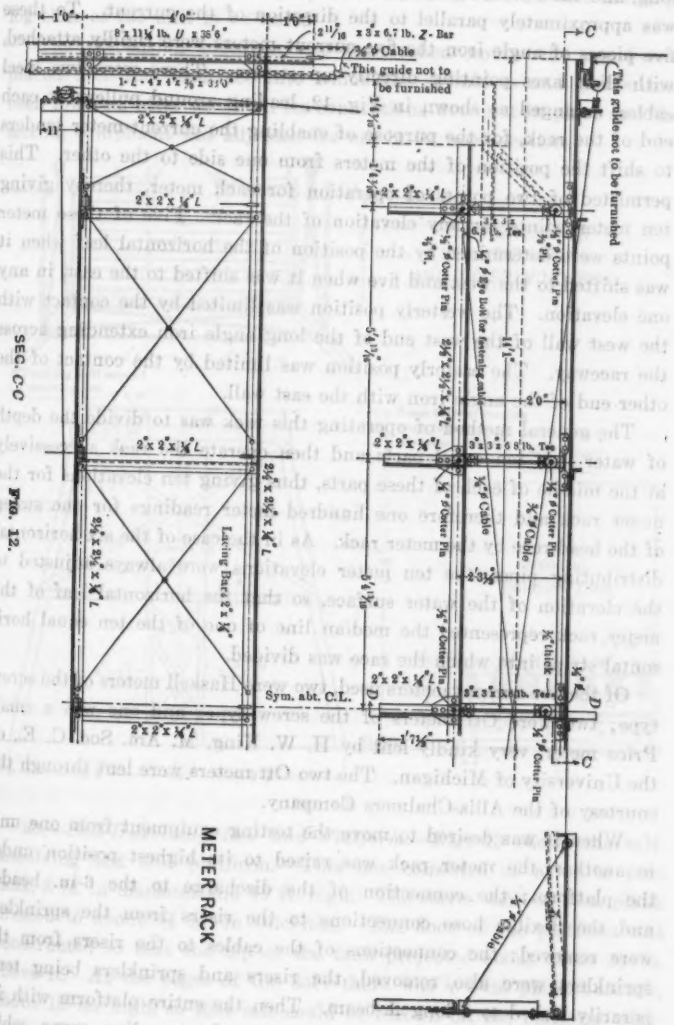


FIG. 12.

METER RACK

long, and these were always parallel to the axis of the head-race, which was approximately parallel to the direction of the current. To these five pieces of angle iron the five current meters were rigidly attached, with their axes pointing directly up stream. There were two steel cables, arranged as shown in Fig. 12, leading around pulleys at each end of the rack, for the purpose of enabling the current-meter tenders to shift the position of the meters from one side to the other. This permitted of two points of operation for each meter, thereby giving ten meter points for any elevation of the rack. Five of these meter points were determined by the position of the horizontal leaf when it was shifted to the west and five when it was shifted to the east, in any one elevation. The westerly position was limited by the contact with the west wall of the west end of the long angle iron extending across the raceway. The easterly position was limited by the contact of the other end of the angle iron with the east wall.

The general method of operating this rack was to divide the depth of water into ten equal parts and then operate the rack successively at the middle of each of these parts, thus giving ten elevations for the meter rack and therefore one hundred meter readings for one sweep of the head-race by the meter rack. As in the case of the six horizontal distributing pipes, the ten meter elevations were always adjusted to the elevation of the water surface, so that the horizontal leaf of the meter rack represented the median line of one of the ten equal horizontal strips into which the race was divided.

Of the five current meters used, two were Haskell meters of the screw type; two were Ott meters of the screw type; and one was a small Price meter, very kindly lent by H. W. King, M. Am. Soc. C. E., of the University of Michigan. The two Ott meters were lent through the courtesy of the Allis-Chalmers Company.

When it was desired to move the testing equipment from one unit to another, the meter rack was raised to its highest position under the platform; the connection of the discharge to the 6-in. header and the flexible hose connections to the risers from the sprinklers were removed; the connections of the cables to the risers from the sprinklers were also removed, the risers and sprinklers being temporarily lashed to a long I-beam. Then the entire platform with its equipment was shifted to one side, using the traveling crane, while the I-beam carrying the sprinklers and current-meter guides was being

moved by the crane for the purpose of lowering its load into the stop-log slots of the unit to be tested.

The suspension platform was then taken up by the crane and placed over the head-race into which the sprinklers and guides had been lowered, after which all its connections were put in place and the testing equipment was adjusted for use in its new location.

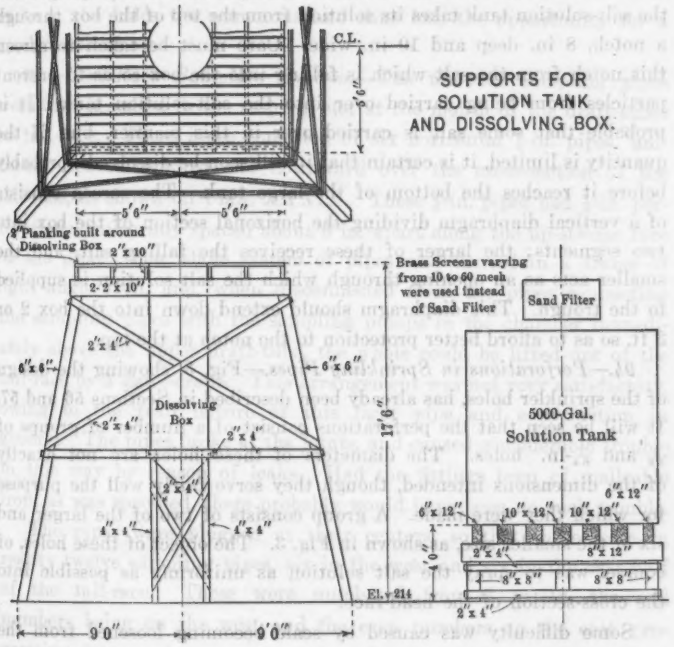


FIG. 13.

93.—Salt-Dissolving Box and Platform.—Fig. 13 shows the salt-dissolving box and platform. This box consisted of a white pine tank, 3 ft. in diameter and 10 ft. high. The staves were of 2-in. plank dressed to about $1\frac{7}{8}$ in. in thickness. The floor of the platform was constructed so that the top of the tank projected about 10 or 12 in. above it. At the edges of the floor there was a 2-in. plank bulwark, about 18 in. high, so that salt could be thrown on the platform without rolling off. In constructing another such platform, the writer would place the dissolving box eccentrically, relative to the platform,

as it is not necessary for the men to stand on opposite sides of the box. With such a scheme, the size of the platform might be made somewhat smaller, and at the same time more convenient. In making use of this box, an ordinary sand screen, of about ¼-in. mesh, is placed over its top opening in such a manner that when salt is thrown on the screen it will fall through and into the box. The trough leading to the salt-solution tank takes its solution from the top of the box through a notch, 8 in. deep and 10 in. wide. Care must be taken to screen this notch from the salt which is falling into the box so as to prevent particles from being carried over into the salt-solution tank. It is probable that some salt is carried over in this manner, but, if the quantity is limited, it is certain that it will soon be dissolved—probably before it reaches the bottom of the large tank. The screen consists of a vertical diaphragm dividing the horizonal section of the box into two segments; the larger of these receives the falling salt, and the smaller acts as an opening through which the salt solution is supplied to the trough. This diaphragm should extend down into the box 2 or 3 ft. so as to afford better protection to the notch at the top.

94.—Perforations in Sprinkling Pipes.—Fig. 3, showing the design of the sprinkler holes, has already been described in Sections 56 and 57. It will be seen that the perforations consist of a number of groups of $\frac{5}{32}$ and $\frac{7}{64}$-in. holes. The diameters of these holes are not exactly of the dimensions intended, though they serve fairly well the purpose for which they were made. A group consists of two of the larger and six of the smaller size, as shown in Fig. 3. The object of these holes, of course, was to spray the salt solution as uniformly as possible into the cross-section of the head-race.

Some difficulty was caused by scale becoming loosened from the interior surface of the pipe system and plugging the holes. It would seem desirable, therefore, to have the holes sufficiently large to eliminate this trouble. Holes ¼ in. in diameter would serve this purpose well, but in any case it would be advisable to have a T on each sprinkler pipe with a perfectly tight valve on the extremity, provision being made for the operation of the valve from above at such times as it might be desired to flush out the pipe system. This was not done in the tests described, but it is suggested as a safeguard in other tests. No serious difficulty, however, was caused by the plugging of the holes after the pipes had been thoroughly flushed once or twice

and blown out by compressed air. It must be carefully borne in mind, in designing sprinklers, that the centrifugal pump should be properly proportioned to project the jets far away from the orifices, for the purpose of reaching all parts of the cross-section with salt solution. It would be preferable, therefore, to have fewer holes, thus creating a larger head for the injection of salt solution, rather than to have the holes too numerous and thus lose the opportunity for a good distribution.

95.—Tail-Race Sampling System.—The tail-race sampling pipes were first installed in the stop-log slot at the mouth of the draft-tubes or tail-race. These pipes consisted of six horizontal ¼-in. pipes, uniformly spaced upward and downward over the cross-section of the tail-race, as shown on Plate XXXIX. These ¼-in. pipes had ⅛-in. perforations, uniformly spaced about 6 in. apart along the up-stream face of the pipe. The whole system of pipes was carried in a frame of light angle iron, and, when disconnected from the pipes connecting the sampling pipes with the sampling pumps in the chamber immediately above the upper draft-tube, the whole could be lifted out of the tail-race by a hand-crane. This arrangement was not very satisfactory, owing to the frail nature of this light pipe and construction in general. The pipes broke at the joints, and caused considerable trouble in this way by reason of leaks. Had the fittings been of malleable iron, as was specified, there probably would have been no such trouble.

The pipes were plugged at their centers, so that there were in reality twelve sampling pipes, six in the west and six in the east half of the tail-race. These were numbered from 1 to 12, the odd numbers being on the west and the even numbers on the east, progressing downward.

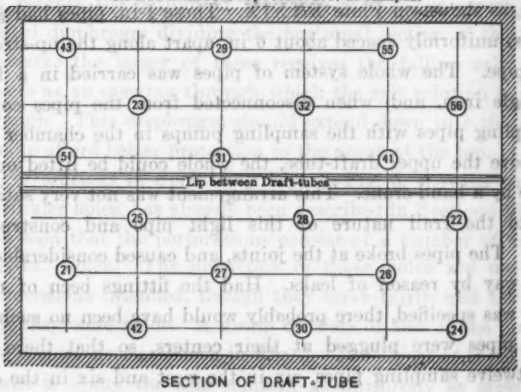
It gradually became evident that the sampling-pipe system could be improved by moving the whole back to the plane of the 4-in. pipes, which had been provided during the construction of the power-house for the admission of Pitot tubes into the tail-race at the mouths of the draft-tubes. This plane is about 8 ft. farther up stream, and has the advantage that the suction pipes leading to the sampling pumps do not pass through a layer of water which was not discharged from the turbines being tested, as was the case at the lower section, where the risers from the sampling pipes passed up through 6 or 8 ft. of

water which was swept back against the power-house wall and probably contained little or no salt other than that naturally in the river.

The caisson compartment shown in the lower part of Plate XXXIX was merely a suggestion as to how the samples might be taken by gravity without the necessity for sampling pumps. This caisson was neither constructed nor used in the tests.

The battery of eighteen sampling pumps stood in the chamber above the upper draft-tube, and was driven by a 10-h.p. motor. It will be described more completely in Section 97.

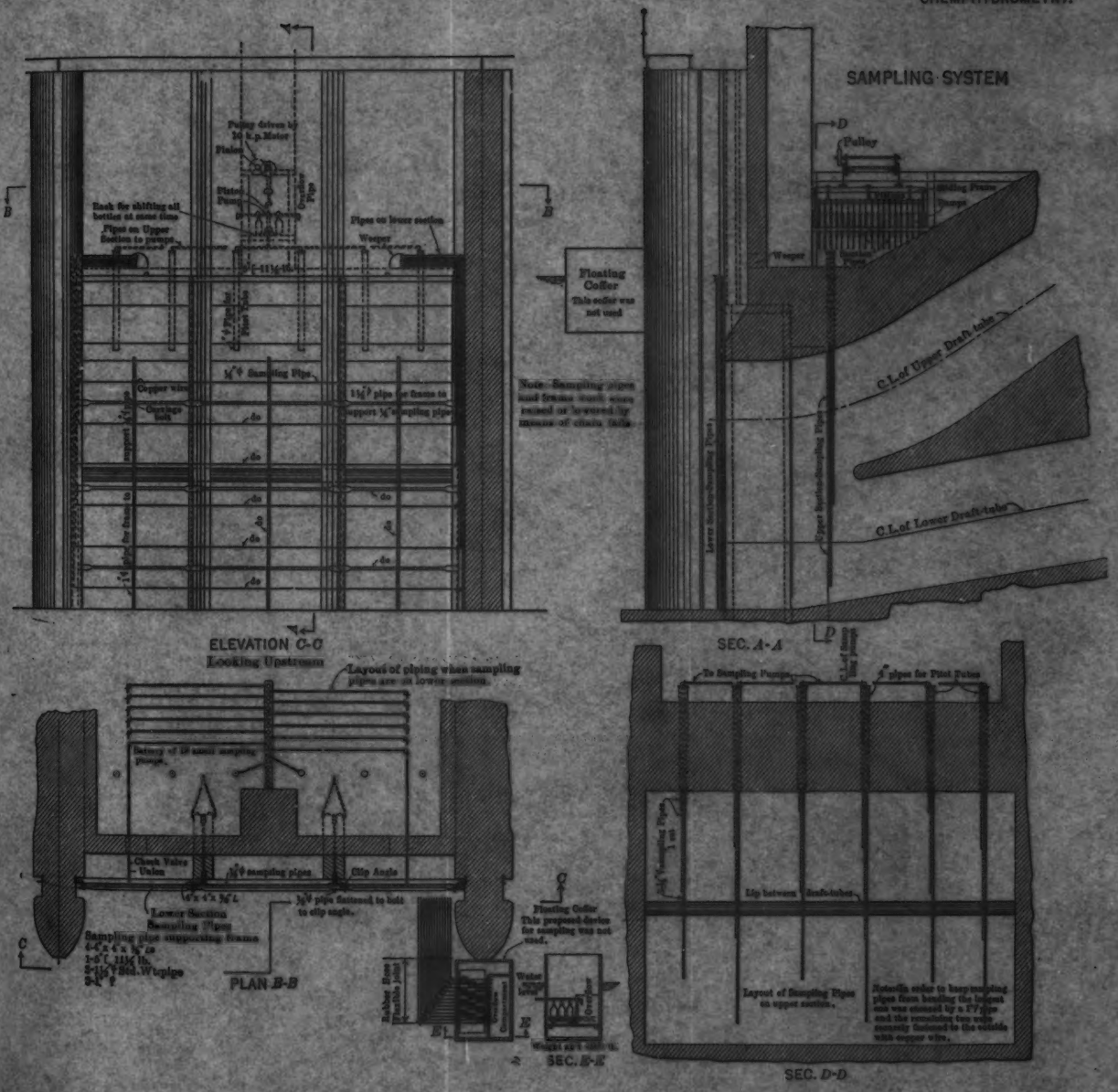
SKETCH SHOWING LOCATION
OF SAMPLES TAKEN IN TAIL-RACE
ON UPPER SECTION.
Circled numbers represent No. of sample.



SECTION OF DRAFT-TUBE
LOOKING UP STREAM.

FIG. 14.

Fig. 14 shows the relative locations, in the upper sampling section, of the intakes of the eighteen sampling pipes which extended down through the 4-in. Pitot tube admission inlets. The numbers of the samples were the same as those shown in Fig. 14. It appears that there was no particular system in connecting up the sampling pipes with the sampling pumps, so that the numbers are not arranged as systematically as they are in the case of the sampling system at the lower section. However, the sample bottles were carefully numbered so as to correspond to the particular locations to which the same numbers are attached in the illustration, so that no ambiguity could result as


SAMPLING SYSTEM
ELEVATION C-C
Looking Upstream
Layout of piping when sampling pipes are on lower section.
PLAN B-B
SEC. A-A
SEC. E-E
SEC. D-D
Floating Collar
This collar was not used
Note—Sampling pipes and frames must were raised or lowered by means of chain falls
Pulley
C.L. of Upper Draft-tube
C.L. of Lower Draft-tube
Lower Section Sampling Pipes
Sampling pipe supporting frame
Floating Collar
This proposed device for sampling was not used.
Layout of Sampling Pipes on upper section.
Note—In order to keep sampling pipes from bending the largest one was encased by a 3" pipe and the remaining two were securely fastened to the outside with copper wire.

to the location in the tail-race from which a particular sample was taken.

96.—Relation of Testing Plant to Turbines.—Plate XL shows sections of the water passages for the turbine unit being tested, and also the head-race and penstock in which the turbines were placed. The upper and lower sections for the tail-race sampling system are also indicated in the longitudinal section. The relative location of the dissolving box and solution tank is also shown.

97.—Photographs of the Testing Equipment.—Fig. 15 is a general view showing the arrangement of salt-solution tanks, salt-distributing system, and current-meter suspension frame.

The salt was brought to the plant by rail, in carloads of 20 to 30 tons, and stored in bins at the west end of the gate-house beyond the extreme right of Fig. 15. The total capacity of the bins was about 100 tons, but probably they did not contain more than 80 or 90 tons at any one time. The hatchways of the bins were immediately under the traveling crane, so that the salt was readily handled by this crane in connection with a second crane at the junction of the gate-houses of the old and new power-houses.

The salt was carried by the crane in a large skip, having a capacity of slightly more than 1 ton. This was lifted to the platform of the dissolving box, where it was dumped. Two men then shoveled the salt rapidly into the box while the circulating pump took water from the city main, pumped it up through the dissolving box and over into the salt-solution tank through the 8 by 10-in. trough previously described.

A 3-in. supply pipe led the salt solution from the tank to the suction of the 8-in. centrifugal pump, at the left center of Fig. 15, immediately under the shed.

A section of the 3-in. pipe can be seen at the point where it enters the 8-in. suction of the centrifugal pump. The back of the vacuum gauge on the 3-in. pipe is also visible. The centrifugal pump can be seen, and also the wooden belt-guard leading from the pump to the motor.

The elevation of the water surface in the canal was evidently about 195.7 ft., so that the suction lift of the centrifugal pump was about 21.3 ft., the center of the pump being approximately at Elevation 217.

In Fig. 18 the salt-solution gauge reader can be seen on the gauge reading scaffold at the side of the gauge-board attached to the salt-solution tank. Two of the head-water sampling pumps, with their suction pipes leading down into the canal, are visible at the center of Fig. 15. Several of the 1-gal. sampling bottles are also visible beside the sampling pumps. One of the gauge-boards can be seen attached to the concrete wall of the entrance to the head-race of the turbine being tested. The head-water was not read at this cross-section, but at a point 5 ft. down stream from the trash racks, which are not visible in Fig. 15. The gauge-board indicates an elevation of water surface of about 195.7. There was another gauge on the opposite wall, and the two were used for the purpose of giving the elevation of the canal water at the current-meter section on each side.

The current-meter suspension frame and platform is at the left center, back of the centrifugal pump and shed. The 8-in. riser supplying the header on top of the platform, and also the top of the header itself, can be seen at the back part of the framework. The two winches carrying the differential pulleys at the ends of the platform, as well as the winch for the current-meter rack in the center of the platform, can be seen.

Fig. 16 shows the salt-dissolving box and salt-solution tank. The iron pipette for calibrating the tank stands between the tank and the right-hand leg of the frame supporting the salt platform. It consisted of a length of 12-in. pipe held in a four-legged wooden frame. The 2-in. pipe carrying the water glass was not attached to the top of the iron pipette at the time of taking the photograph. The 10-ft. gauge-board carrying the 1-in. glass tube in a slot along its center, together with the rubber tube attaching the bottom of the glass gauge to the bottom of the tank, is at the right of the salt-solution tank. The gauge reader's scaffold stands at the right of the gauge-board. The 3-in. riser leading from the circulating pump to the bottom of the salt-dissolving box is shown, also the pipe leading under the planks from the city water supply at the left. This pipe joins the pipe leading to the bottom of the salt-dissolving box, and may be disconnected from it by a gate-valve. A portion of the crane which assists in handling salt is at the extreme right.

Fig. 17 illustrates the operation of charging the salt-solution tank. A skip-load of salt has just been raised into position by the crane and

PLATE XL.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXX, No. 1385.
GROAT ON
CHEMI-HYDROMETRY.

RELATION
OF
TESTING PLANT TO TURBINES

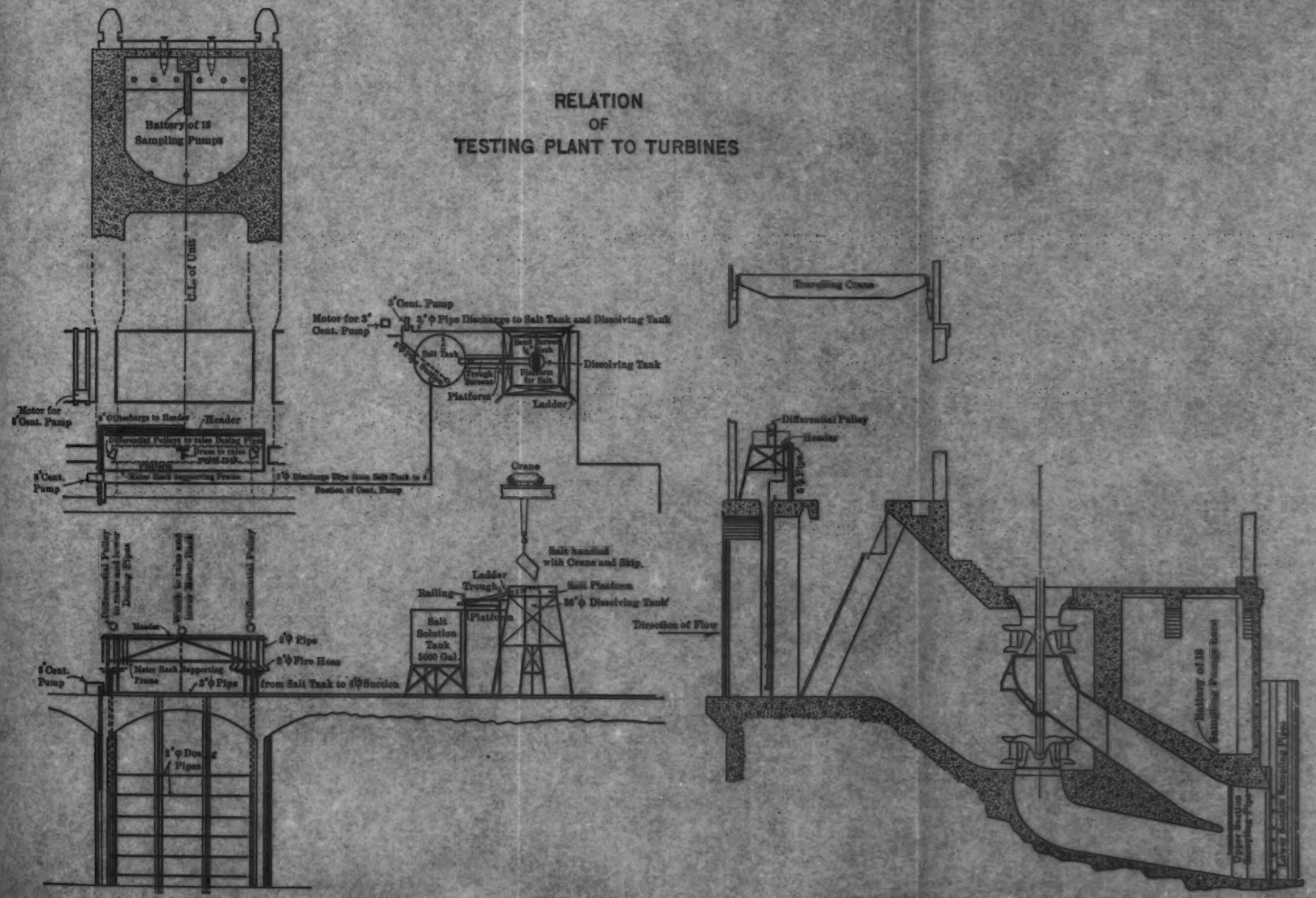
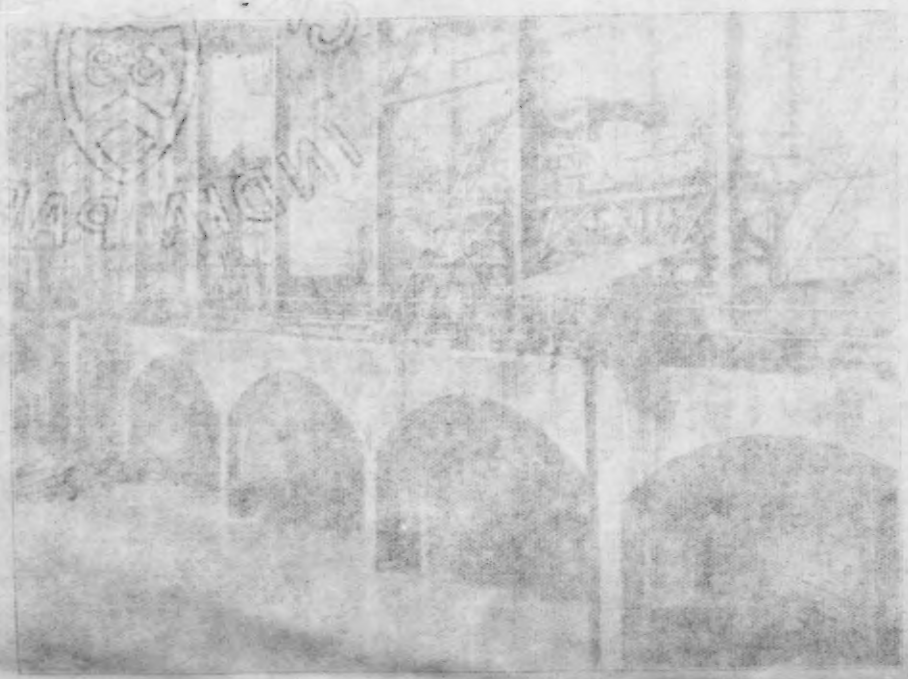


FIG. 15.—SALT-SOLUTION SYSTEM AND CURRENT-METER SUSPENSION FRAME FROM EAST BANK OF CANAL.



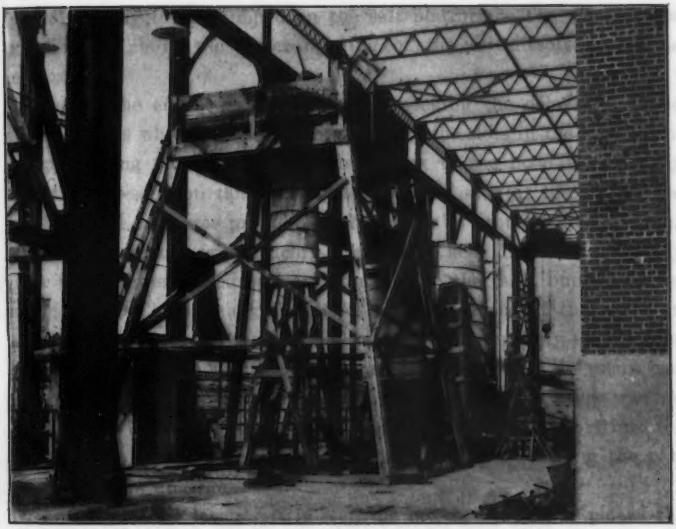


FIG. 16.—SALT-DISSOLVING BOX AND SALT-SOLUTION TANK.

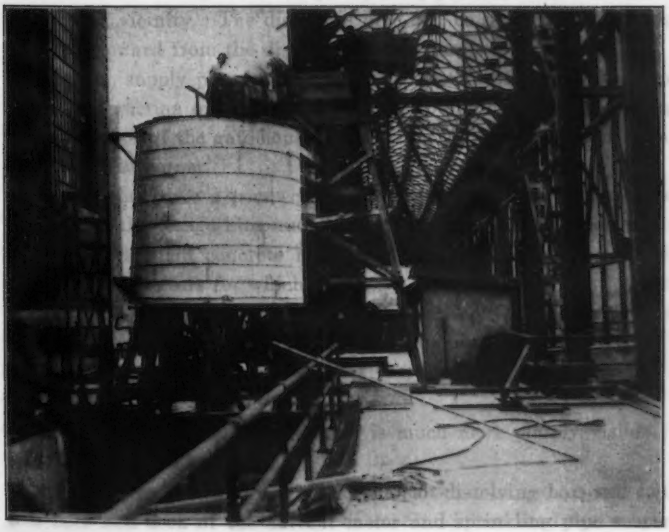
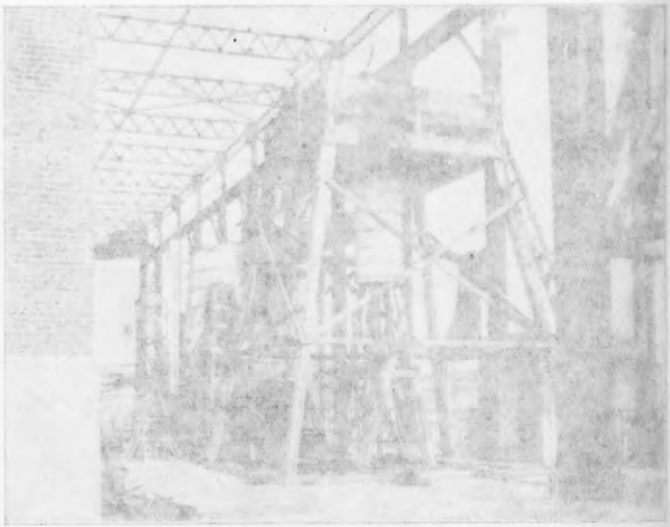


FIG. 17.—CHARGING SALT-SOLUTION TANK.



is about ready to be dumped on the salt platform. The sand screen immediately above the upper opening of the salt-dissolving box is in view.

One of the engineers may be seen knocking the débris from one of the screens which he has withdrawn from the slots in the 8 by 10-in. trough leading from the salt-dissolving box to the hood immediately over the salt-solution tank. This hood can be seen at the end of the trough. The tank was provided with a cover to protect the surface of the water from any falling débris and also to keep off the wind. Very little trouble was caused by waves forming in the tank, but the writer would suggest having a grilled float on the surface of the water in such a tank, in order to still the surface as completely as possible. This was not done during the tests described herein, and the result was that on several occasions the meniscus of the salt-solution gauge was not as steady as it might have been, thus introducing small errors into the calculation of the quantity of salt solution consumed during the particular tests in question.

At the extreme lower left is seen the box in which was placed the motor driving the small circulating pump, which latter is in view but somewhat obscured by the multiplicity of pipes, timbers, cables, and chain in its vicinity. The discharge pipe can be plainly seen leading vertically upward from the discharge of the pump.

The 3-in. supply pipe leading from the bottom of the salt-solution tank to the suction of the 8-in. centrifugal pump can be seen leading across the floor of the gate-house and making a right angled turn near the lower right corner.

Fig. 17 also gives a perspective down through the west gate-house toward the salt-storage bins and the chemical laboratory, which is obscured by the small concrete house enclosing the motors for the ice sluice-gates, the gears for which are shown near the lower right corner.

Fig. 18 gives another view of the salt-solution tank and salt-dissolving box connected by the 8 by 10-in. trough and hood.

The gauge reader may be seen at the left of the tank, making a record of the gauge reading.

The centrifugal circulating pump is much more plainly visible in this than in Fig. 17.

Fig. 19 gives a bird's-eye view of the salt-dissolving box, salt-solution tank and deck of the current meter and sprinkling pipe suspen-

sion platform. The differential pulleys and twelve suspension cables which were attached to the risers from the six sprinkling pipes are at the ends of the platform, and the winch for raising and lowering the meter rack is at the middle of the up-stream side of the platform. The 6-in. header fed by the 8-in. riser from the centrifugal pump may be seen on the deck of the frame. The 8-in. centrifugal pump is under the shed at the extreme lower right of Fig. 19. The left center foreground shows the tops of the racks in one of the units tested.

The screen for salt and a quantity of salt piled on the salt platform are in view.

Fig. 20 shows the centrifugal pump driven by a 75-h.p. motor, and the 8-in. discharge leading to the 8-in. riser and header on the platform. Parts of the twelve 2-in. distributing pipes leading from the header, six at each end, may be seen. The lever for operating the four head-water sampling pumps, the cross-head connecting the four plungers, the four plungers themselves, and the bottom parts of the suctions of these small pumps can be seen just above a collapsed section of 2-in. hose pipe leading from one of the distributing pipes to the sprinkler system. These sampling pumps were small hand-pumps supplied by the Deming Company, and the handles can be seen just above the cross-head and under the hand-lever.

Fig. 21 shows the iron pipette calibrator used in the calibration of the large salt-solution tank. The top and bottom of this calibrator are not plainly visible, but they were each conical in shape. The cone at the top was for the purpose of allowing the escape of air through the 2-in. pipe at the top when the calibrator was filled with water; the conical bottom was to facilitate draining the calibrator when it was emptied through the throttle valve at the vertex of the cone. The water-glass at the side of the 2-in. pipe projecting above the vertex of the upper cone was for the purpose of ascertaining the level of the water in the calibrator. A paper label can be seen pasted on this gauge. The label carried a mark at the proper elevation for a given capacity. The volume of water discharged from the calibrator was about 240 liters (8.466 cu. ft.) when it had been previously filled up to the mark with canal water.

The method of calibrating the tank was to set this calibrator down on the top of the rack platform, shown in Fig. 19, so that the water in

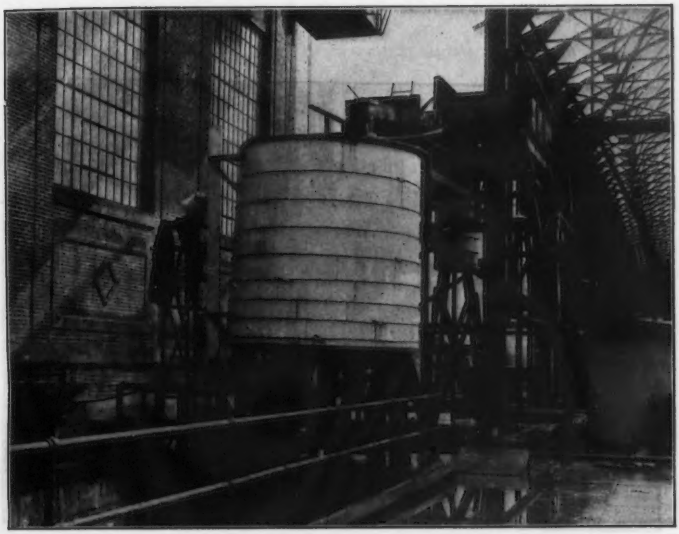


FIG. 18.—SALT-SOLUTION TANK, AND OBSERVER RECORDING READINGS OF SALT-SOLUTION GAUGE.

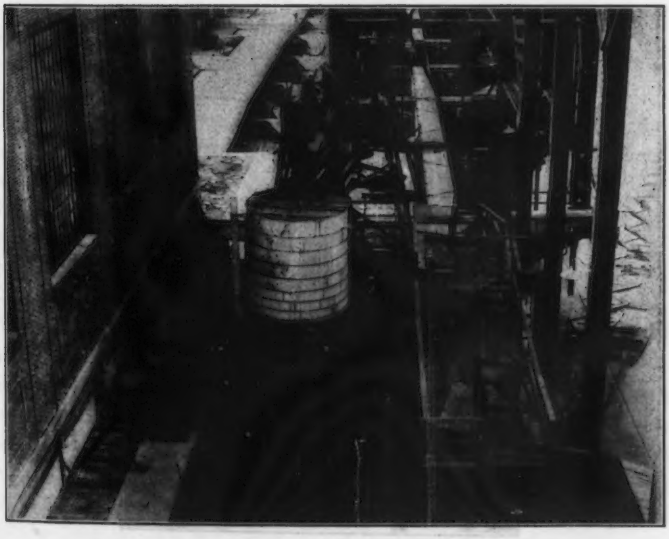
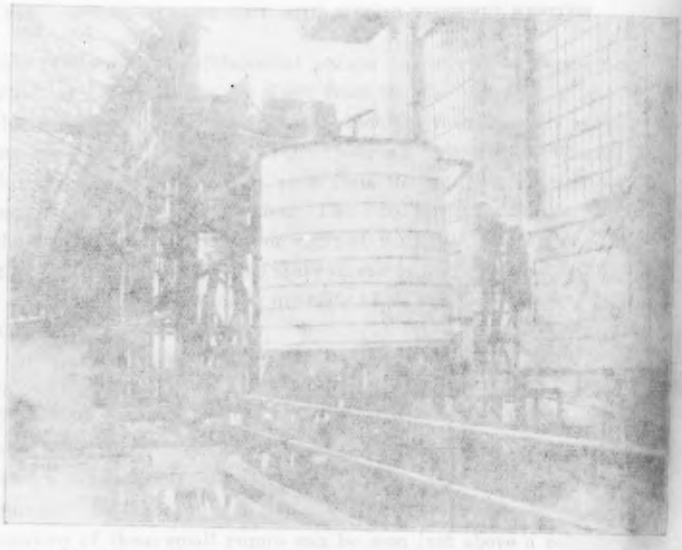
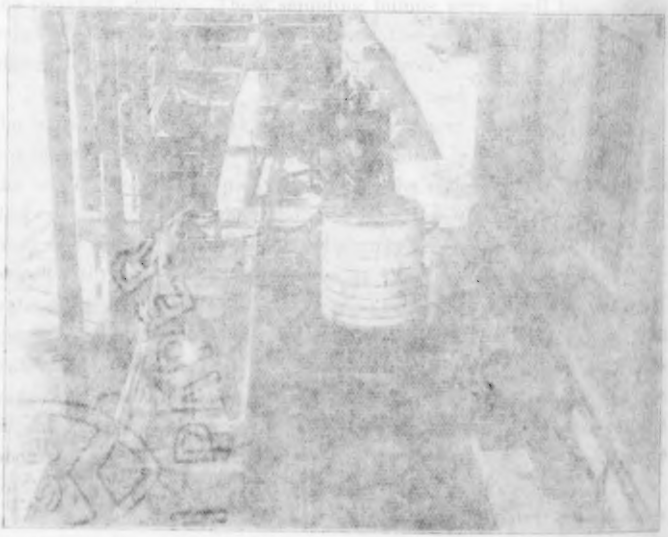


FIG. 19.—BIRD'S-EYE VIEW OF SALT-SOLUTION TANK AND DISSOLVING BOX.





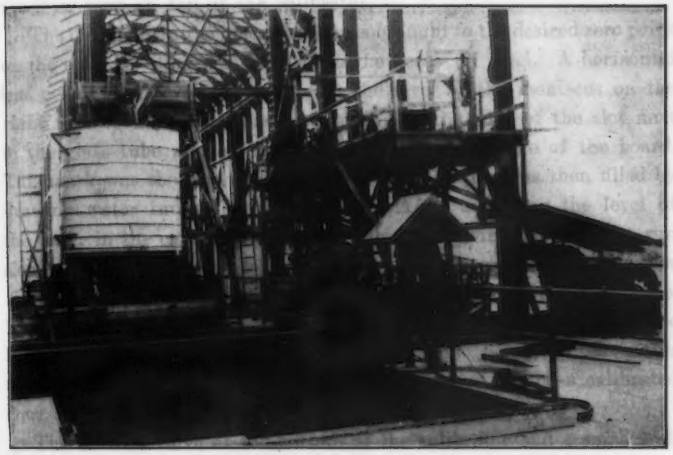


FIG. 20.—CENTRIFUGAL PUMP DRIVEN BY 75-H. P. MOTOR, AND 8-IN. DISCHARGE LEADING TO 8-IN. RISER AND HEADER ON PLATFORM.

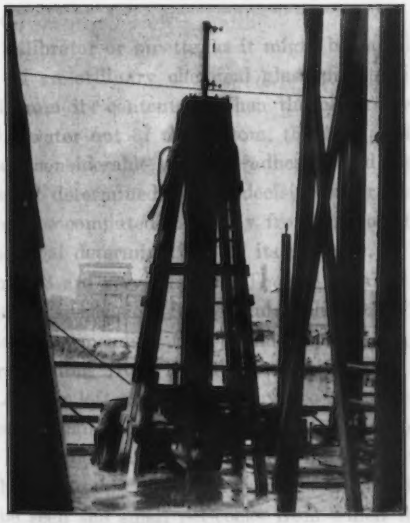
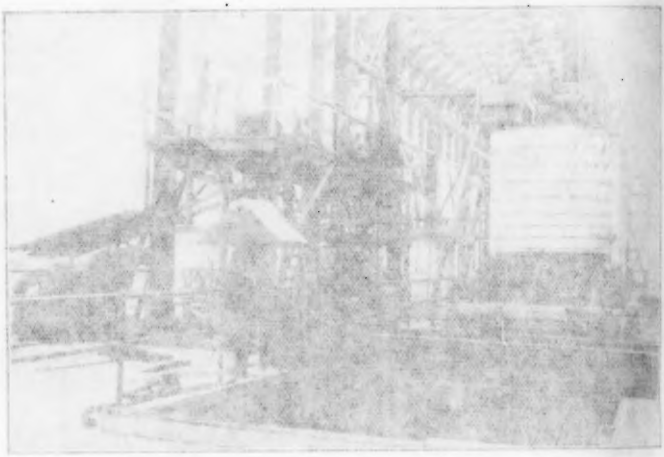


FIG. 21.—LARGE IRON CALIBRATOR. THE CONICAL TOP AND BOTTOM ARE NOT VISIBLE.



the salt-solution tank could be drawn by gravity through a hose into the 2-in. pipe on top of the calibrator.

The meniscus in the glass gauge was brought to the desired zero point on the scale by filling the tank up to the necessary level. A horizontal lead-pencil mark was then made at the level of the meniscus on the right face of the gauge-board, beginning at the edge of the slot next to the glass tube and extending toward the outer edge of the board. This mark was the zero graduation. The calibrator was then filled by drawing water into it from the tank. After adjusting the level of the water in the calibrator very carefully by allowing the water to run slowly from the tank into the calibrator, when nearly full, another mark was made at the lower level of the meniscus in the glass tube. The calibrator was then emptied by opening the throttle valve at the bottom, the water supply from the tank, of course, being cut off. After this the process was repeated until the gauge-board had been calibrated down to the desired limit.

The throttle valve at the bottom of the calibrator had a small leak which was taken care of by slipping a bottle under the calibrator so as to capture the water which escaped by drops. By pouring this captured water back into the calibrator at the proper moment the error was eliminated.

This large calibrator or pipette, as it might be called, possessed the characteristic of an ordinary chemical glass pipette, in that its delivery differed from its contents. When the calibrator is discharged by drawing the water out of the bottom, the sides and angles of the calibrator retain considerable water by adhesion and capillarity. This difference was not determined by any decisive experiments during the tests, but it may be computed indirectly from the results of the gravimetric and chemical determinations of its capacity.

Figs. 22 and 23 are very similar. They give fairly good views of the current-meter suspension frame and platform. The differential pulleys and cables suspending the sprinkler system may be seen, and also the risers leading from the 6-in. header on the deck over the framework. At the left center one may see the gate-valves on the risers, the pressure gauges and elbows connecting by means of nipples with the 2-in. hose pipes which form the flexible joint between the rigid header and the adjustable sprinkler system. On the bottoms of the nipples may be seen the small pet-cocks from which the salt-solution

samples were taken when it was desired to ascertain how uniformly the salt solution was being distributed by the 8-in. centrifugal pump. There were twelve of these sampling cocks, and, as has been previously stated, the samples taken therefrom were absolutely uniform in all tests made for the purpose of studying the distribution of salt solution among the twelve risers.

The table on which the five current-meter observers made their records is in place, and the telephone, enabling communication with all the other observers connected with the test, is at the lower right center just above the right end of the table.

Figs. 22 and 23 give a better view of the layout of the 8-in. discharge, 8-in. riser, and 6-in. header than any of the other illustrations.

Fig. 24 shows the discharge from the uppermost sprinkler when slightly raised above the surface of the water in the head-race. It will be seen that there is a system of holes discharging vertically upward, another discharging horizontally up stream and a third discharging vertically downward. Each of the six sprinkler pipes was perforated in this manner, so as to give as perfect a mixture in the head-race as possible.

The scaffold, on which one of the engineers may be seen, was arranged for the purpose of facilitating the current-meter work. It was found more convenient to attach and displace the meters when working on this platform than to attempt to raise and lower the meter rack through the gate-slot in the floor of the gate-house. The current-meter rack may be seen immediately back of the platform.

Fig. 25 shows the current-meter rack carrying the five current meters, as in a regular test. The rack is raised sufficiently to elevate the meters above the water surface.

Fig. 26 shows the battery of eighteen tail-race sampling pumps connected to the frame to which was attached the gear driving the cross-head which actuated the plungers of the pumps. The gears were driven by a 5-h.p. motor belted to the pulley shown at the top. Nine of the pumps discharged toward the observer and nine in the opposite direction, so that nine bottles were placed on one side of the pumps and nine on the other. These sample bottles were placed on a board provided with cleats to prevent the bottles from slipping. One of these boards can be seen in the lower foreground connected with two levers. This device was used to shift the bottles simultaneously from under

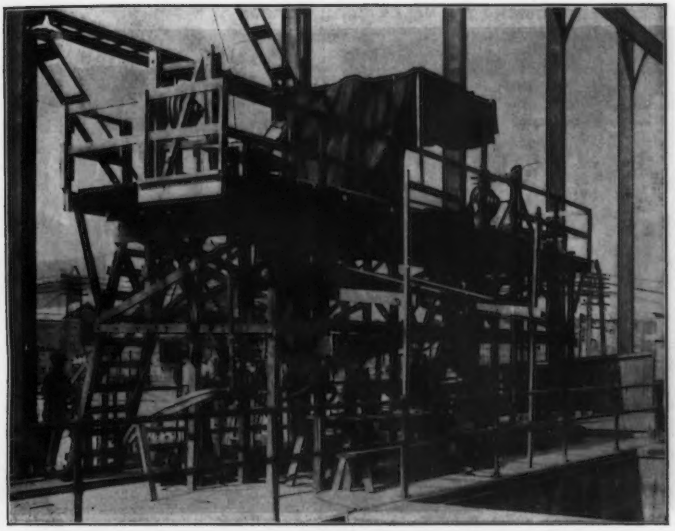


FIG. 22.—CURRENT-METER SUSPENSION FRAME AND PLATFORM, WHICH ALSO CARRIES THE DISTRIBUTING-PIPE SUSPENSION SYSTEM AND THE 6-IN. HEADER.

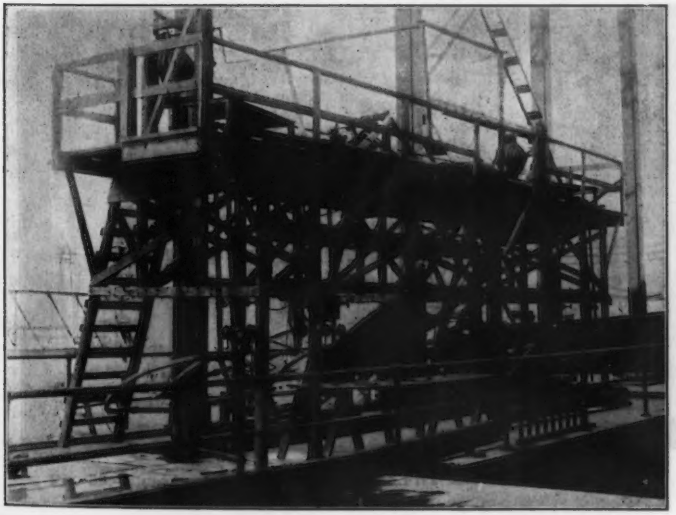
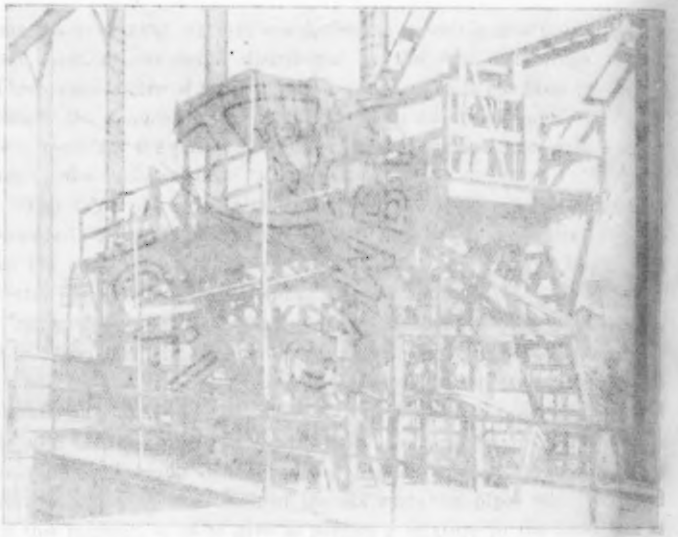


FIG. 23.—CURRENT-METER SUSPENSION FRAME AND PLATFORM.



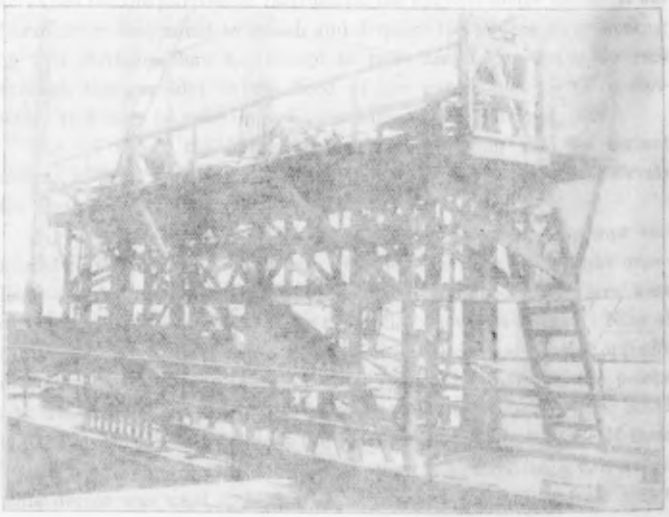


FIG. 24.—DISCHARGE OF UPPERMOST SPRINKLER OF SALT-DISTRIBUTING SYSTEM.

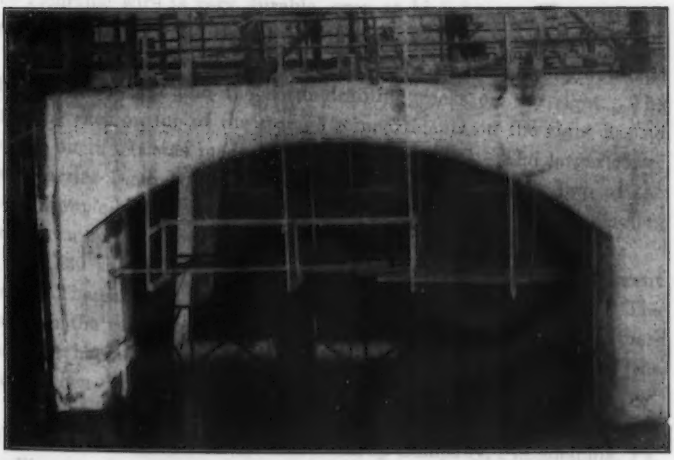


FIG. 25.—CURRENT-METER RACK, CARRYING FIVE METERS READY FOR TEST.

the discharges of the pumps when it was desired to stop the sampling. The levers were shifted in the opposite direction when it was desired to start the sampling. One attendant on each side of the battery of pumps took care of the sampling and watched the filling of bottles and the operation of pumps, so as to correct any interference with the proper taking of samples due to the failure of a pump or overflowing of a bottle.

Prior to the time of starting the samples, the attendants gauged each of the pumps, so that each bottle would be three-fourths full by the end of the test. The bottles were of the ordinary 1-gal. spring water type.

Fig. 27 shows the battery of separatory funnels and water-baths carrying the casseroles into which the separatory funnels discharge when the attendant opens the valve slightly so as to admit a small quantity of sample into a particular casserole.

The water-baths were heated by especially constructed electric heaters, the electrical switches for which may be seen on the wall back of the water-baths.

Fig. 28 shows one of the asbestos-board trays made to suport a 14 by 28-in. water-bath over the coil of nichrome resistance wire shown in the bottom of the tray. These heaters were very satisfactory, and the nichrome wire is very durable, even at high heats. The writer is indebted to Mr. G. K. Herzog for this design.

Fig. 29 is a view in the laboratory, showing the sample-storage shelves on the left and the bottle-draining racks on the right. The draining rack on top of the storage compartments on the right has no bottles on it. It was made for 1-qt. cream bottles. The larger lower rack carries 1-gal. bottles, a few of which are in place. Two of the 1-gal. sample-bottle carriers and one of the 1-qt. bottle sample carriers are shown on the floor below the draining rack.

Fig. 30 shows the large shunt used in testing the direct-current units. The shunt is in place as it was attached to Unit No. 12. The basin in the center of the shunt contains kerosene into which a thermometer hangs for the purpose of furnishing the temperature of the shunt when desired.

This shunt is of peculiar construction, and was designed by Mr. I. B. Smith, of the Leeds and Northrup Company, Philadelphia, Pa.

Fig. 31 shows the voltmeter and ammeter potentiometers which

were used in determining the direct-current electrical power across the shunt, shown in Fig. 30, and between the terminals of the generator. These instruments were also manufactured by the Leeds and Northrup Company. They are extremely sensitive, and the direct-current power readings are very accurate, probably being within 0.1% of the truth. The instruments are even more accurate than this under laboratory conditions.

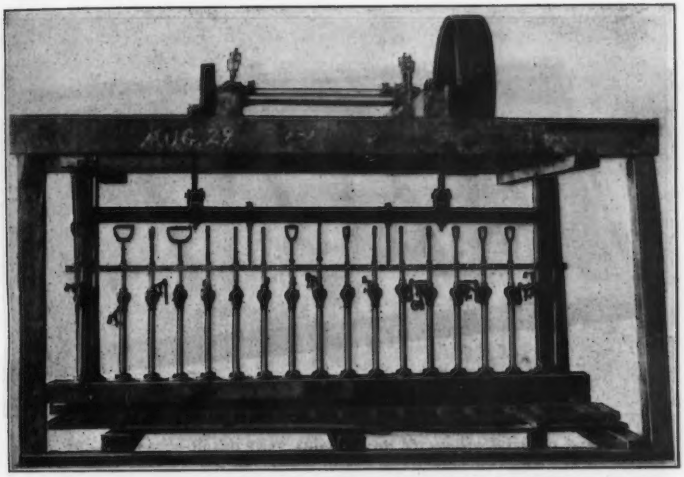
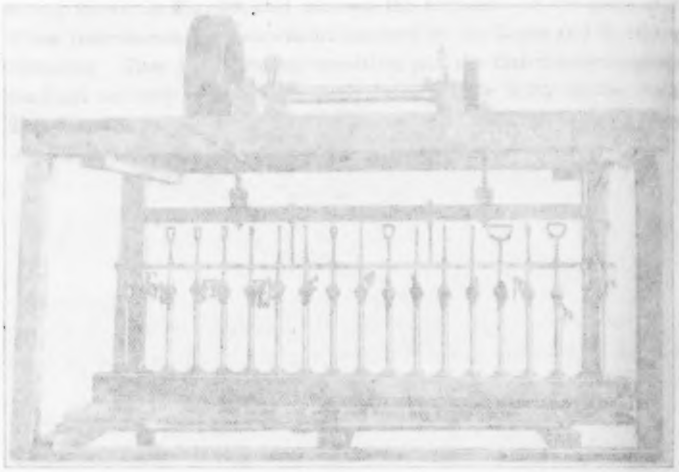


FIG. 26.—BATTERY OF EIGHTEEN TAIL-RACE SAMPLING PUMPS.

FIG. 27.—BATTERY OF SEPARATORY FUNNELS AND WATER-BATHS, SHOWING CASSEROLES IN PLACE FOR CONDUCTING EVAPORATIONS.



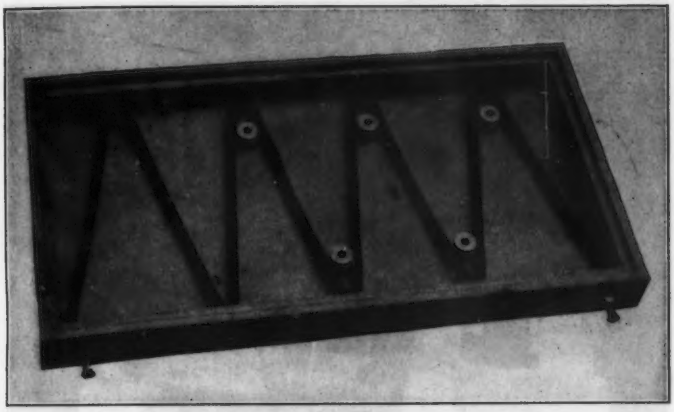
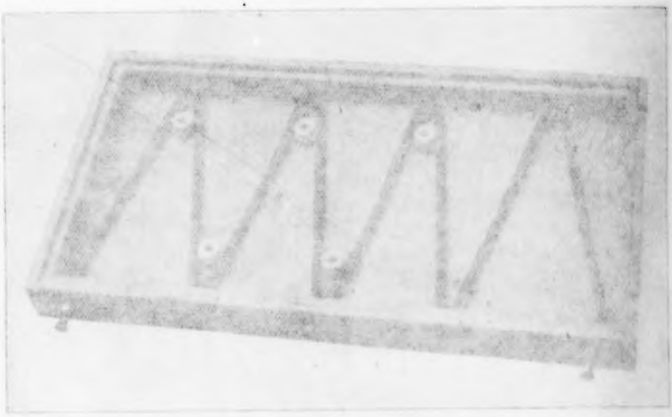
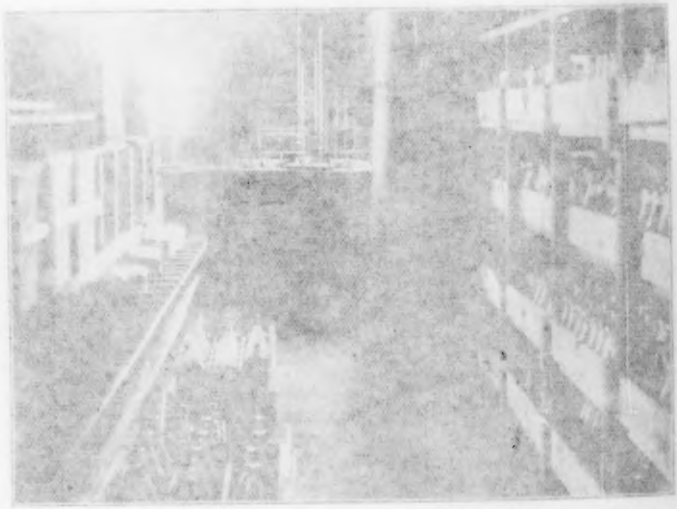


FIG. 28.—ASBESTOS-BOARD TRAY AND COIL OF NICHROME WIRE TO SUPPORT AND HEAT WATER-BATHS.

FIG. 29.—VIEW IN CHEMICAL LABORATORY, SHOWING SAMPLE STORAGE, BOTTLE-DRAINING RACKS, BOTTLE CARRIERS, AND OTHER EQUIPMENT.





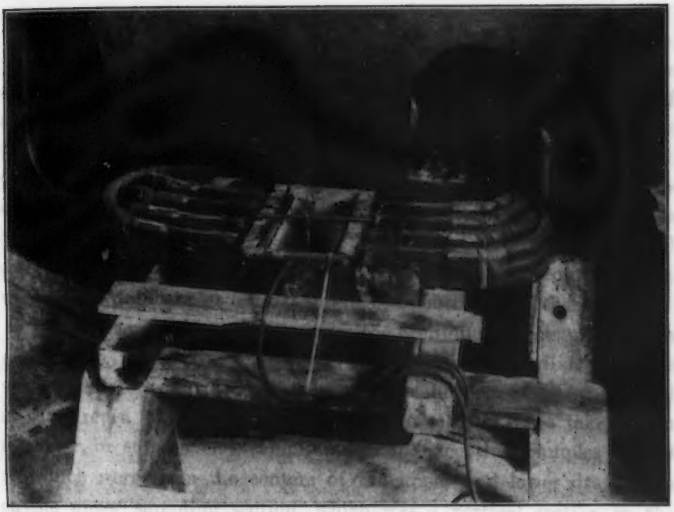


FIG. 30.—10 000-AMPERE SHUNT, DESIGNED BY MR. I. B. SMITH, OF THE LEEDS AND NORTHRUP COMPANY.

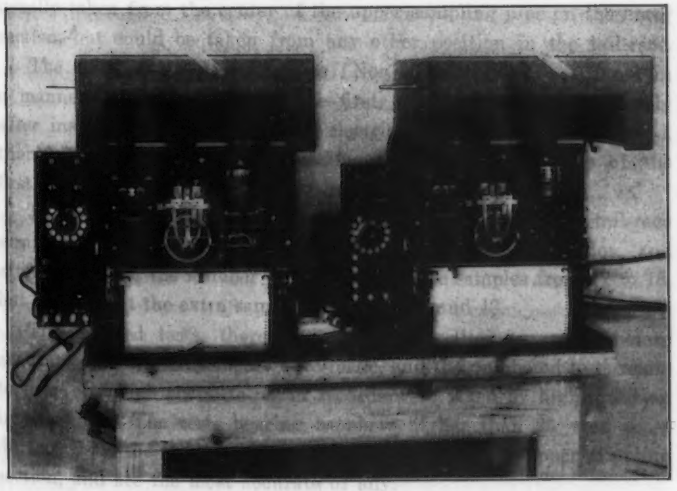
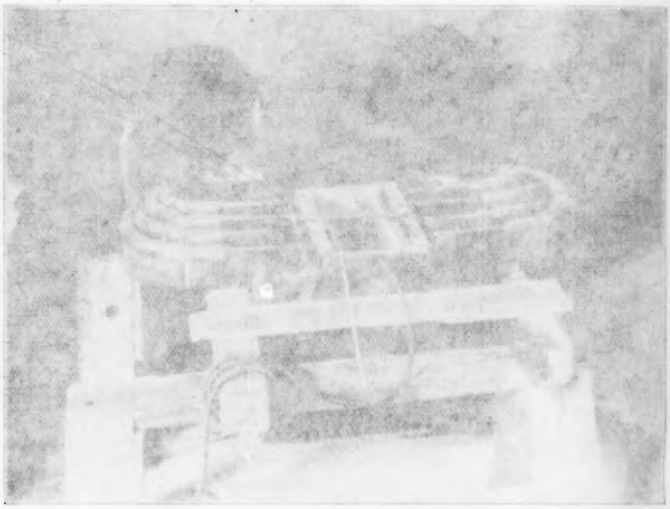
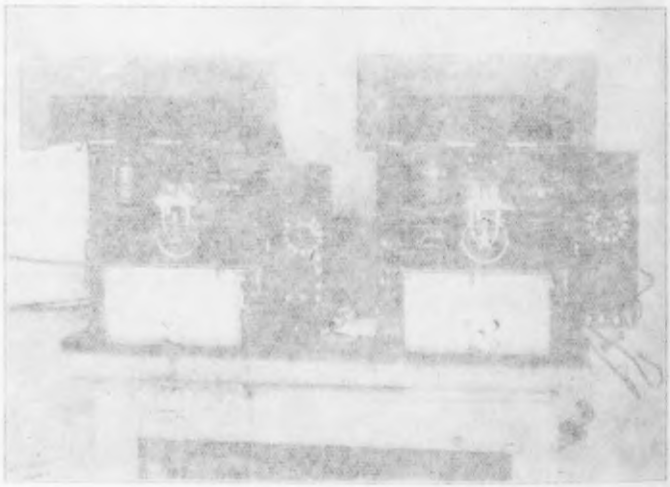


FIG. 31.—VOLTMETER AND AMMETER POTENTIOMETERS FOR MEASURING DIRECT-CURRENT POWER ON TESTS.





PART IV.

THE TESTS.

93.—Tests on Unit No. 13.—The tests on Unit No. 13 may be divided into three series. The first is indicated by letters instead of numbers. The second comprises the tests bearing numbers from 1 to 25, inclusive, and the third comprises all the remaining numbers. Certain tests which were not turbine tests have been given letters, or numbers, and these letters or numbers, therefore, are not in the tables or on the diagrams connected with the turbine tests proper. In all cases the tabulations of turbine tests give them in consecutive order, whether the numbers, or letters, appear consecutively or otherwise.

The first series (letters) comprises the preliminary tests which were made while the corps of observers was becoming familiar with its work, and before the high-precision electric instruments were inserted in the circuit. In this series the tail-race samples were taken from the lower, or stop-log-slot, sampling section, excepting Samples 41 and 42, which were from the centers of the upper and lower draft-tubes on the upper sampling section, which was a vertical section in the plane of the Pitot tube pipes extending into the tail-race just down stream from the discharge ends of the draft-tubes. Sample 43 was usually taken from the center of the upper sampling pipe on the lower section, but could be taken from any other position in the tail-race.

The tests of the second series (Nos. 1 to 25) were executed in a manner similar to those of the first, but were made immediately after inserting the high-precision electric measuring equipment, and, therefore, are to be regarded as more accurate than those of the first series.

The tests of the third series were made after moving the tail-race sampling system from the lower to the upper sampling section, and after increasing the number of regular tail-race samples from 12 to 18, but cutting out the extra samples, Nos. 41, 42, and 43.

The lettered tests, therefore, relate to preliminary work, before accurate power measurements were made, and the numbered tests relate to those in which accurate power records were taken by high-precision instruments. The tests bearing numbers higher than 25 were those during which the tail-race samples were taken from the upper sampling section, and are the most accurate of any.

The object in moving the tail-race sampling system from the lower to the upper section, after the second series, was to secure more reliable samples and a larger number of them. The principal reason for this change was the fact that the risers leading to the sampling pumps had to pass up through several feet of water which was more or less diluted by water from other turbine units not being tested. Should any of these risers leak it would be likely to take in water which did not contain a sufficient quantity of salt or, at least, did not come from the location at which it was intended to secure a sample.

99.—Tests on Unit No. 12.—There was only one series of tests on Unit No. 12, numbered from 200 to 212, inclusive. They were executed in consecutive order, from September 28th to October 13th, 1914, and cannot be considered as good as the third series on Unit No. 13. This is due to the less favorable weather conditions, to changes in personnel of observers and manipulators, and also to the fact that the hydraulic conditions of the tests were not so uniform and satisfactory. In particular, the power measurements are not as satisfactory as those on Unit No. 13, and this is probably due to the fact that the power was observed and the electrical instruments manipulated in the latter case by Mr. I. B. Smith, the designer of the instruments.

100.—Tests on Unit No. 11.—There were two series of tests on Unit No. 11. The tests were started with No. 105, on September 12th, but after running four more tests, ending with No. 109, on September 18th, it became necessary to shut down the unit for repairs, and there were no more until November 9th, when Test 111 was made. This second series was then continued consecutively up to 122 on November 21st, which ended the series, and in fact all the turbine tests.

The generator of this unit was of the alternating-current type, and those of Units Nos. 12 and 13 were both of the direct-current type. Consequently, an entirely new set of electrical measuring instruments was required, with additional observers, and correspondingly more difficulty in securing data. The power records, however, are very consistent, under the circumstances, and approach in accuracy those of the best tests on Unit No. 13.

101.—Field Data.—Tables 46, 47, 48, and 49,* contain a summary of all the field data observed during the entire series of turbine tests,

* These tables are not reproduced in this paper, but are filed in the Library of the Society for reference by any one interested.

exclusive of the data necessary for the chemical work proper. As a general rule, all instrumental readings given on these sheets are averages computed from numerous readings taken at the end of equal intervals of time, with the maximum variation during the entire test recorded immediately after the corresponding averages.

The method of measuring the head was specified in the contract, but a slight divergence therefrom was agreed on, owing to the difficulty of measuring the head-water exactly 5 ft. down stream from the racks at the water surface. This difficulty arose from the fact that the gauge-boards would have been somewhat inconveniently situated at that exact distance in all the tests, as the distance to the gauge-board from the racks at the water level varied considerably with changes in elevation of head-water due to the slope of the racks. At surface elevation 197 the location of the gauge-boards was just about as specified in the contract.

The principal head-water readings, however, were made by measuring downward at three points across the head-race from an angle iron which was fixed at Elevation 204. The scales on the measuring rods were inverted, with the 204-mark at the bottom of the rod, so that, by holding the bottom of the rod in contact with the water surface and reading the inverted scale on the rod at the edge of the angle iron, the true elevation of the water surface was ascertained without further correction.

As a matter of convenience, the rods passed through holes in the flat side of the angle, so that by boring holes the size of a nail at various points along the rod and then inserting a nail in the hole nearest to the proper graduation mark, the rod could be left by the gauge reader to swing loosely at about the correct elevation, thus being ready for use at any time without the necessity for much adjustment.

The three rods and the two gauge-boards gave the elevations of the water surface at five points in the head-race. The distances between these five points were not exactly equal across the race, owing to the desire to obtain readings of the two rods adjacent to the center rod, which would indicate the surface elevation back of the two rack girders where the water surface was relatively low.

The tail-water elevation was observed in much the same manner, there being two gauge-boards at the side-walls and three rods manipulated in a manner similar to that used in the head-water readings. The

three rods, however, were just outside the power-house, in the center-line of the three openings of the tail-race formed by the two side-walls and the two tail-gate girders. The rods, therefore, were equally spaced on each side of the rod at the center.

There was also a float-gauge in the head-race, at the inside of the west rack girder, and one in the tail-race at the west wall. These floats did not give accurate results because of the influence of the velocity heads created in and around the gauge-board box. Probably more accurate results could have been obtained by a further study and re-arrangement of the float-gauges.

The output of Units Nos. 13 and 12 was measured by three different sets of direct-current equipment. The first set consisted of the ordinary switch-board instruments; the second consisted of high-precision instruments purchased from the Leeds and Northrup Company, and the third was a Weston outfit, furnished and operated by the Allis-Chalmers Company for its own information.

In the tests of Unit No. 11 there were also three sets of instruments. The current in this case, however, was 25-cycle, at about 2 200 volts, thus limiting the tests to a fixed speed which exceeded 100 rev. per min.

The first set was the switch-board equipment; the second was a high-precision Weston outfit purchased for the purpose, and the third was the Allis-Chalmers' Weston outfit.

The speed of the turbine was taken by a tachometer, and checked by actual count.

The tables show the data for the salt solution, the temperature of the solution in the tank, the concentration computed from the hydrometer reading, the two pairs of divisions on the gauge scale between which the surface of the solution stood at the start or finish of each test, the time of passing these divisions, the time of starting the salt-solution discharge, the time when the discharge was adjusted and uniform, and the time of starting and finishing the test.

The pressures at which the distributing pipes were operated are given, but it must be understood that these are mere readings of gauges on the sides of the pipes. These gauges had not been calibrated, and were subject to the usual errors when attempting to secure pressures from pipes inserted in the side of another pipe in which the pressures are to be estimated. The gauges, however, served their pur-

pose well in supplying a means for controlling the distribution of salt in the head- and tail-races.

There is also a column for remarks at the right of each of the data tables.

102.—Tables of Observed Concentrations.—Space will not be devoted to the detail of all the concentrations observed in all the tests. If it is desired to examine the detail of any test, which is not fully supplied in this paper, the original data are on file, and access will be furnished by the writer.

The four principal series of titrations, however, given in detail in the next four sections, furnish a complete exhibit of the best of the chemical work.

In the case of Units Nos. 12 and 13, and the first series of tests on Unit No. 11, the strength of the silver nitrate solution was about 1.56 grammes per liter. In the case of the second series of tests on Unit No. 11, in fact, the last of the turbine tests, the strength was about 1.45 grammes per liter, resulting approximately from the solution of 1.45 grammes in 1 liter of distilled water.

As a general rule, the tests were titrated the day after they were made, the evaporations having been made over night in the laboratory, which operated continuously. The second treatments of certain of the tests, usually indicated by the letter *B* affixed to the regular test number, are exceptions to this rule. They were made long after the first treatments, and the results show that the samples suffered more or less from reductions of volume due to evaporation during the interval.

This experience indicates that extraordinary care must be taken to preserve samples, when they are to be kept for any length of time before treatment. One-quart large-mouthed cream bottles with paraffined cork stoppers will not do for this purpose.

103.—Concentrations Observed During Eleven Consecutive Tests on Unit No. 13.—The dilute salt solution is prepared by diluting one 10-c.c. pipette of salt solution to the graduation mark of a 1-liter flask with distilled water, 10 c.c. of this dilution then being taken for titration. In all cases $\frac{1}{2}$ liter of normal head-water and $\frac{1}{2}$ liter of tail-water are evaporated to about 10 c.c. in a casserole for titration. Titrations in parentheses have been omitted from the calculations as doubtful. The unit of titration is $\frac{1}{100}$ c.c., all decimal points being omitted except in the final averages, where the unit is 1 c.c. The

capacities of the pipettes and flasks mentioned are nominal; the actual capacities may be determined by Figs. 4 and 6. In all cases, Pipette No. 18 791 was used on strong salt solution, and Pipette No. 21 047 on dilute salt solution; and Flasks 1 456 and 1 476 were used on dilution of salt solution. Flask No. 11 was used for measuring normal head- and tail-water in all cases except Test 32, on which Flask No. 3 was used. Missing test numbers represent tests which were not chemical. The chemical tests follow in the consecutive order in which they were made.

CONCENTRATIONS OBSERVED DURING ELEVEN TESTS ON UNIT NO. 13.

Titrations in parentheses have been omitted as doubtful. The unit of titration is $\frac{1}{1000}$ c.c., all decimal points being omitted except in final averages, where the unit is 1 c.c.

TEST 26, AUGUST 26TH, 1914.

TITRATION. NITRATE (1.56 GRAMMES), AUGUST 24th.			
Dilute salt solution. Temp. 19½° cent.		Normal head-water. Temp. 18° cent.	
5 280	5 280	1 310	1 305
5 290	5 290	1 305	1 306
5 290	5 288	(1 330)	1 295
5 287	5 285	1 300	1 295
5 285	5 283	1 282	1 310
5 282	5 290	1 295	(1 325)
Mean = 52.858		Mean = 13.003	

TAIL-WATER TITRATIONS. Temp. 18½° cent.

5 810		5 813		5 770			
	5 871				5 805		5 860
5 840		5 925		5 920			
	5 365			5 440		5 481	
5 425		5 380			5 465		
	5 395			5 408		5 420	

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper....	58 518	58 388	58.46
Lower....	53 912	54 428	54.199
Averages.	56.215	56.408	56.329 = Arith. mean.
Weighted mean = 56.276			

TEST 32, AUGUST 26TH, 1914.

TITRATION	NITRATE (1.56 GRAMMES), AUGUST 24th.		
Dilute salt solution. Temp. 19.8° cent.		Normal head-water. Temp. 19° cent.	
5 248	5 257	1 302	1 305
5 260	5 245	1 310	1 313
5 235	5 260	1 290	1 292
5 255	5 254	1 310	1 302
5 260	5 253	1 300	1 300
5 255	5 262	1 295	1 300
Mean = 52.537		Mean = 13.016	

TAIL-WATER TITRATIONS. Temp. 19° cent.

5 290		5 360		5 405			
	5 405				5 410		5 470
5 290		5 400		5 450			
	5 325			5 367		5 395	
5 320		5 380			5 350		
	5 460			5 370		5 392	

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper.....	53 490	54 337	53.867
Lower.....	53 712	53 748	53.732
Averages..	53.601	54.042	53.799 = Arith. mean.
Weighted mean = 53.776			

TEST 33, AUGUST 27TH, 1914.

TITRATION. NITRATE (1.56 GRAMMES). AUGUST 24th.			
Dilute salt solution. Temp. 19.8° cent.		Normal head-water. Temp. 21° cent.	
5 342	5 345	1 290	1 280
5 348	5 350	1 308	1 300
(5 315)	5 350	1 288	1 293
5 340	5 340	1 293	1 282
5 337	5 340	1 305	1 290
5 340	5 340	1 290	1 293
Mean = 53.429		Mean = 12.927	

TAIL-WATER TITRATIONS. Temp 21.8° cent.							
5 570		5 637		5 740			
	5 685		5 657		5 870		
5 700		5 730		5 715			
	5 510			5 568		5 608	
5 585		5 558			5 597		
	5 560			5 550		5 592	

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper....	56 644	57 455	57.004
Lower....	55 526	55 830	55.698
Averages.	56.085	56.642	56.351 = Arith. mean.
Weighted mean = 56.299			

TEST 39, AUGUST 28TH, 1914.

TITRATION. NITRATE (1.56 GRAMMES). AUGUST 24th.			
Dilute salt solution. Temp. 23.8° cent.		Normal head-water. Temp. 19° cent.	
5 390	5 390	1 297	1 310
5 380	5 388	1 315	1 310
5 400	5 400	1 325	1 325
5 400	5 395	1 300	1 300
5 400	5 398	1 298	1 300
5 400	5 400	1 295	1 310
Mean = 53.951		Mean = 13.071	

TAIL-WATER TITRATIONS. Temp. 19° cent.							
5 847		5 990		5 880			
	6 160		5 995		6 060		
6 190		6 160		6 135			
	5 333			5 335		5 465	
5 330		5 305			(5 070)		
	5 365			5 325		5 340	

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper....	60 694	60 175	60.463
Lower....	53 332	53 663	53.498
Averages.	57.013	56.622	57.185 = Arith. mean.
Weighted mean = 56.968			

104.—Concentrations Observed During the Tests on Unit No. 12.—The dilute salt solution is prepared in the same way as described in Section 103. In these tests ½ liter of normal head-water, ½ liter of tail-water and ½ liter of special dilution were evaporated to about 10 c.c. in a casserole for titration. Titrations in parentheses have been omitted as doubtful. The unit of titration is $\frac{1}{100}$ c.c., all decimal points being omitted except in the final averages, where the unit is 1 c.c. The capacities of the pipettes and flasks mentioned are nominal; the actual capacities may be determined by Figs. 4 and 6. In all cases, Pipette No. 18 791 was used on strong salt solution, and Pipette No. 21 047 on

dilute salt solution, or stock solution for special dilutions. Flasks were used as indicated in the record of each test.

TEST 40, AUGUST 28TH, 1914.

TITRATION.		NITRATE (1.56 GRAMMES). AUGUST 24th.	
Dilute salt solution. Temp. 21° cent.		Normal head-water. Temp. 22° cent.	
5 330	5 330	1 340	1 333
5 327	5 339	(1 425)	1 325
5 343	5 325	1 328	1 298
5 320	5 335	1 320	1 310
5 330	5 333	1 325	1 310
5 330	5 332	1 312	1 290
Mean = 53.312		Mean = 13.174	

TAIL-WATER TITRATIONS. Temp. 20.5° cent.

(5 320)	5 610	5 455	5 550	5 555	5 570
5 573		5 650		5 610	
5 480	5 460	5 495	5 500	5 553	5 600
	5 500		5 515		5 570

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper.....	55 720	55 712	55.716
Lower.....	54 888	55 476	55.192
Averages..	55.279	55.594	55.439 = Arith. mean.

Weighted mean = 55.402

TEST 41, AUGUST 31ST, 1914.

TITRATION. SEPTEMBER 3d.		NITRATE (1.56 GRAMMES). AUGUST 31st.	
Dilute salt solution. Temp. 24.5° cent.		Normal head-water. Temp. 20° cent.	
5 135	5 150	1 300	1 315
5 140	5 147	1 302	1 325
5 160	5 155	1 317	1 330
5 135	5 140	1 320	1 315
5 163	5 140	1 330	1 330
5 150	5 150	1 320	1 319
Mean = 51.471		Mean = 13.186	

TAIL-WATER TITRATIONS. Temp. 20° cent.

5 320	5 445	5 392	5 385	5 440	5 494
5 345		5 470		5 485	
5 405	5 385	5 380	5 405	5 430	5 325
	5 390		5 420		5 455

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper.....	53 944	54 510	54.196
Lower.....	53 900	54 070	53.994
Averages..	53.922	54.290	54.095 = Arith. mean.

Weighted mean = 54.063

105.—Concentrations Observed During the First Series of Tests on Unit No. 11.—The dilute salt solution is prepared by diluting one 10-c.c. pipette of salt solution to the graduation mark of a 1-liter flask with distilled water, 10 c.c. of this dilution then being taken for titration, except in Tests 108 and 109, where four pipettes of salt solution were diluted to 3 liters. In all cases ½ liter of normal head-water, ½ liter of tail-water and ½ liter of special dilution are evaporated to about 10 c.c. in a casserole for titration. Titrations in parentheses have been

omitted as doubtful. The unit titration is $\frac{1}{1000}$ c.c., all decimal points being omitted except in final averages, where the unit is 1 c.c. Capacities of pipettes and flasks mentioned are nominal; the actual capacities may be determined by Figs. 4 and 6. In all cases Pipette No. 18 791 was used on strong salt solution, and Pipette No. 21 047 on dilute salt solution, or stock solution for special dilutions. Flasks were used as indicated in the record of each test.

TEST 42, AUGUST 31ST, 1914.

TITRATION.		NITRATE (1.56 GRAMMES). AUGUST 31ST.	
Dilute salt solution. Temp. 29° cent.		Normal head-water. Temp. 21⅛° cent.	
5 465	5 465	1 320	1 320
5 455	5 455	1 300	1 320
5 465	5 470	1 345	1 335
5 455	5 460	1 315	1 320
5 460	5 463	1 315	1 305
5 465	5 455	1 300	1 300
Mean = 54.611		Mean = 13.163	

TAIL-WATER TITRATIONS. Temp. 20.5° cent.					
5 485	5 590		5 630		
5 620		5 620		5 685	
5 515	5 660		5 693		
5 587	5 650		5 585		5 705
	5 590	5 590	5 625		
			5 665		5 650

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper....	55 740	56 570	56.109
Lower....	56 042	56 460	56.274
Averages.	55.891	56.515	56.192 = Arith. mean.
Weighted mean = 56.127			

TEST 43, SEPTEMBER 1ST, 1914.

TITRATION. SEPTEMBER 3D. 4TH.		NITRATE (1.56 GRAMMES). AUGUST 31ST.	
Dilute salt solution. Temp. 26° cent.		Normal head-water. Temp. 25° cent.	
5 425	5 420	1 310	(1 260)
5 430	5 425	1 307	1 300
5 422	5 423	1 315	1 305
5 425	5 425	1 320	1 315
5 425	5 425	1 312	1 315
5 425	5 425	1 305	1 315
Mean = 54.246		Mean = 13.108	

TAIL-WATER TITRATIONS. Temp. 23.5° cent.					
5 530	5 620		5 780		
5 670	5 730	5 815	5 690		5 880
			5 785		
None*	5 400		5 490		5 535
	5 390	5 420	5 480	5 510	5 490

* Bottle broken.

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper.....	56 730	57 838	57.222
Lower.....	54 033	55 010	54.644
Averages..	55.382	56.424	56.009 = Arith. mean.
Weighted mean = 55.786			

106.—Concentrations Observed During the Second Series of Tests on Unit No. 11.—The dilute salt solution is prepared by diluting one

10-c.c. pipette of salt solution to the graduation mark of a 1-liter flask with distilled water, 10 c.c. of this dilution then being taken for titration. In all cases ½ liter of tail-water and ½ liter of special dilution are evaporated to about 10 c.c. in a casserole for titration. Titrations in parentheses have been omitted as doubtful. The unit titration is $\frac{1}{1000}$ c.c., all decimal points being omitted except in the final averages, where the unit is 1 c.c. Capacities of pipettes and flasks mentioned are nominal; the actual capacities may be determined by Figs. 4 and 6. Flask Nos. 1 407 and 1 457 were used for dilution of salt solution on all tests except Test 117, in which Flasks Nos. 1 407 and 1 476 were used.

TEST 44, SEPTEMBER 1ST, 1914.

TITRATION.		NITRATE (1.56 GRAMMES). AUGUST 31ST.	
Dilute salt solution. Temp. 28° cent.		Normal head-water. Temp. 23° cent.	
5 445	5 440	1 300	1 295
5 440	5 435	1 310	1 282
5 450	5 445	1 300	1 310
5 450	5 445	1 287	1 305
5 450	5 447	1 300	1 300
5 450	5 445	1 280	1 300
Mean = 54.452		Mean = 12.974	

TAIL-WATER TITRATIONS. Temp. 22° cent.					
5 475		5 675	5 685	5 745	5 875
	5 755			5 900	
5 633		5 845			
5 650	5 655	5 644	5 645	5 742	5 765
	5 615		5 670		5 730

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper....	56 766	58 012	57.320
Lower....	56 410	57 104	56.795
Averages.	56.588	57.558	57.058 = Arith. mean.

Weighted mean = 56.952

TEST 45, SEPTEMBER 2D, 1914.

TITRATION.		NITRATE (1.56 GRAMMES). AUGUST 31ST.	
Dilute salt solution. Temp. 22° cent.		Normal head-water. Temp. 25.3° cent.	
5 450	5 460	1 303	1 300
5 460	5 460	1 300	1 300
5 450	5 472	1 296	1 294
5 450	5 465	1 286	1 294
5 456	5 450	1 290	1 290
5 452	5 455	1 290	1 290
Mean = 54.567		Mean = 12.944	

TAIL-WATER TITRATIONS. Temp. 24° cent.					
5 362		5 450	5 446	5 555	5 590
	(5 085)			5 565	
5 427		5 545			
5 365	5 394	5 400	5 472	5 485	5 640
	5 380		5 440		5 475

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper....	54 460	55 390	54.925
Lower....	53 848	55 024	54.501
Averages..	54.154	55.207	54.701 = Arith. mean.

Weighted mean = 54.549

TEST 46, SEPTEMBER 2D, 1914.

TITRATION, SEPTEMBER 4TH. NITRATE (1.56 GRAMMES). AUGUST 31ST. CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

Dilute salt solution Temp. 19.5° cent.		Normal head-water. Temp. 21.5° cent.	
5 122	5 133	1 295	1 297
5 135	5 133	1 291	1 275
5 130	5 132	1 275	1 275
5 130	5 130	1 270	1 270
5 140	5 132	1 270	1 303
5 140	5 135	1 274	1 285
Mean = 51.327		Mean = 12.817	

	West.	East.	Averages.
Upper.....	56 422	56 605	56.503
Lower.....	53 905	54 084	54.004
Averages..	55.164	55.344	55.254 = Arith. mean.

Weighted mean = 55.227

TAIL-WATER TITRATIONS. Temp. 21.5° cent.

5 540		5 649		5 592	5 590		5 690
5 575	5 672	5 775			5 770		
	5 370			5 394		5 415	
5 335		5 422			5 395		
	5 435			5 428		5 410	

Flasks Nos. 1 407 and 1 457 were used for special dilutions in all tests. Pipettes Nos. 9 171 and 9 206 were used on strong salt solution, dilute salt solution, and stock solution. No normal head-water was evaporated. The methods of making the solutions are explained in the rules for the chemical laboratory at the end of Part II. See Sections 88 and 89.

107.—Discharge Computations.—Tables 50, 51, 52, and 53 (Plates XLI, XLII, XLIII, and XLIV), contain all the data necessary for the chemical determination of the discharge in the various tests. These calculations have been carried out with more refinement, in certain respects, and more elaborately, than necessary. This is especially true when these methods are compared with the simple one of weighing samples, explained in Sections 10 to 18, and applied in Section 78 to the determination of the capacity of the iron calibrator.

In view of the full explanation of the theories and equations given in Parts I and II, it will not be necessary to enter into a detailed explanation of the nature of the discharge computations, which are, in fact, quite fully explained by the tables.

TEST 200, SEPTEMBER 28TH, 1914.

TITRATION, SEPTEMBER 29TH.
Nitrate (1.56 grammes), September 25th.

Dilute salt solution. Temp. 15.75° cent. Flask No. 1 456.		Normal head-water. Temp. 15° cent. Flask No. 11.	Special dilutions.* Temp. 15° cent. Flask No. 1 456.		
4 685	4 675	1 300	4 350	4 316	8 666
4 687	4 685	1 288	4 410	4 324	8 734
4 687	4 685	1 308	4 377	4 345	8 722
4 677	4 670	1 290	4 335	4 350	8 685
4 675	4 680	1 290			
4 683	4 671	1 310			
Mean = 46.800		Mean = 12.977	Mean = 43.509		

TAIL-WATER TITRATIONS.
Temp. 15.5° cent. Flasks Nos. 3–11.

4 420		4 390		4 587	
	4 454		4 525		4 614
4 516		4 510		4 575	
	4 830		5 337		5 345
5 130		†5 465		5 540	
	5 447		5 610		5 540

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper.....	44 580	45 752	45.101
Lower.....	52 180	54 744	53.604
Averages.	48.380	50.248	49.353 = Arith. mean.

Weighted mean = 48.962.

* 10 c.c. salt solution from each of Samples ———, 40 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

† Average of the two values to the right and the lower one on the left. Sample pipe was broken.

TEST 201, SEPTEMBER 29TH, 1914.

TITRATION, OCTOBER 1ST.
Nitrate (1.56 grammes), September 9th.

Dilute salt solution. Temp. 15.5° cent. Flasks Nos. 1 456–1 476.		Normal head-water. Temp. 14.5° cent. Flask No. 11.	Special dilutions.* Temp. 15° cent. Flasks Nos. 1 456–		
5 150	5 148	1 295	4 958	4 932	9 890
5 149	5 148	1 295	4 937	4 901	9 838
5 145	5 145	1 300	4 875	4 920	9 795
5 145	5 145	(1 250)	4 870	4 915	9 785
		1 290			
Mean = 51.469		Mean = 12.950	Mean = 49.135		

TAIL-WATER TITRATIONS.
Temp. 15° cent. Flask No. 3.

5 297		5 310		5 382	
	5 301		5 386		5 375
5 297		5 310		5 382	
	5 455		5 615		5 580
5 400		†5 562		5 607	
	5 455		5 615		5 580

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE

	West.	East.	Averages.
Upper	53 030	53 813	53.378
Lower	54 680	55 994	55.410
Averages..	53.855	54.904	54.394 = Arith. mean.

Weighted mean = 54.140.

NOTE: Depths 1 and 3 averaged chemically.
" 4 and 6 " "

* 10 c.c. salt solution from each of Samples ———, 30 c.c., diluted with 2 100 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

† Average of the two values to the right and the lower one on the left. Sample pipe was broken.

TABLE 50.—Data for Computation of Turbine Discharge. Compiled from Records of Tests made on Unit No. 13, July 18th, 1914, to

Line.	UNIT No. 12.	Formula.	Units.	Symbol.	G	H	I	K	L	N	O	S	T
1	Temperature—Salt solution in tank during test				18.9	19.1	19.9	19.9	19.9	17.9	17.9	18.6	18.4
2	Salt solution when pipetted in laboratory for dilution with distilled water				21.0	20.0	21.0	20.0	20.0	23.5	23.0	23.0	24.0
3	Distilled water for diluting salt solution in laboratory				21.0	20.0	21.0	24.0	24.0	23.5	23.0	23.0	23.5
4	Normal head-water when measured for evaporation in laboratory				21.0	20.0	21.0	22.0	23.5	23.0	23.0	23.0	23.0
5	Tail-water when measured for evaporation in laboratory				21.0	19.5	21.0	21.0	22.5	23.0	23.5	23.5	24.0
6	Head-water in canal				19.1	19.5	19.4	19.4	19.4	19.6	19.6	19.3	19.6
7	Tail-water in canal				19.1	19.5	19.4	19.4	19.4	19.6	19.6	19.3	19.6
8	Tail-water in tail-race				No. 18 791 9.980	No. 18 791 9.979	No. 18 791 9.980	No. 18 791 9 985	No. 18 791 9 984	No. 18 791 9.989	No. 18 791 9.983	No. 18 791 9.985	No. 18 791 9.988
9	Mean temperature of head and tail-water during tests	(Line 6 + Line 7) ÷ 2	c.c.		19.1	19.5	19.4	19.4	19.4	19.6	19.6	19.3	19.6
10	Number of pipette used in measuring out salt solution for dilute salt solution				1 456	1 479	1 479	1 476	1 476	1 474, 1 476	1 476	1 456	1 456, 1 476
11	Volume of salt solution discharged by pipette for dilute salt solution at temperature of Line 2		c.c.		1 042.67	1 008.50	1 008.58	1 008.60	1 008.58	1 008.86	1 008.55	1 008.60	1 008.58
12	Numbers of flasks used in measuring distilled water				No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047
13	Volume of flask measuring distilled water at temperature of Line 2		c.c.		10.015	10.012	10.013	10.016	10.016	10.015	10.015	10.020	10.020
14	Number of pipette used in measuring out dilute salt solution		c.c.		54.656	54.594	54.470	55.165	55.238	53.515	52.606	54.573	55.105
15	Volume of dilute salt solution discharged by pipette at temperature of Line 2		c.c.		5 458.50	5 448.90	5 440.80	5 509.10	5 513.00	5 345.30	5 252.90	5 446.40	5 502.60
16	Titration per liter of dilute salt solution in laboratory		c.c.		100.49	100.53	100.55	100.58	100.58	100.54	100.54	100.45	100.50
17	Ratio of dilution of salt solution with distilled water		c.c.		548 280	547 440	547 070	558 930	554 170	537 210	532 120	547 080	563 760
18	Titration per liter of salt solution at temperature of Line 2	Line 15 ÷ Line 14 Line 13 × Line 10 Line 16 × Line 17	gms.		1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622
19	Concentration of silver nitrate, grammes per liter, approximately		gms.		294.7	294.3	294.2	297.3	297.3	288.7	284.0	294.2	302.4
20	Concentration (approximate) in grammes per liter of salt solution at temperature of Line 2												
21	Approximate strength of solution, in percentage, at temperature of Line 2	$p = \frac{c}{d}$ (from curve)			24.55	24.33	24.33	23.05	23.13	24.43	24.09	24.57	25.44
22	Density of salt solution in tank during test at temperature of Line 1				1.1874	1.1872	1.1879	1.1838	1.1836	1.1844	1.1844	1.1874	1.1922
23	Density of salt solution when pipetted for dilution with distilled water in laboratory at temperature of Line 2				1.1841	1.1854	1.1850	1.1869	1.1852	1.1850	1.1794	1.1837	1.1898
24	Density of salt solution at mean temperature of head and tail-water, temperature of Line 9				1.1870	1.1866	1.1866	1.1868	1.1851	1.1836	1.1811	1.1875	1.1922
25	Titration of salt solution at mean temperature of head and tail-water, temperature of Line 9 (per liter)	Line 18 × Line 24 ÷ Line 22	c.c.	v	548 960	547 580	547 450	556 250	555 503	538 282	532 670	548 640	564 170
26	Gauge reading on salt solution tank at start				B	C	C	C	C	10	10	10	10
27	Gauge reading on salt solution tank at end				50	50	50	50	43	72	71	68	70
28	Total time, in seconds, elapsed between divisions at start and end				81	81	82	82	82	2 438.6	2 389.1	2 418.9	2 393.0
29	Rate of discharge; actual	$\frac{3\,466 \times (\text{Line } 27 - \text{Line } 26)}{\text{Line } 28}$	sec.		973.4	972.9	973.0	1 027.0	946.1				
30	Equivalent rate of discharging salt solution at mean temperature of head and tail-water, temperature of Line 8	Line 29 × Line 25 ÷ Line 24	c.f.s.	q	0.02394	0.02394	0.02137	0.02545	0.02669	0.02613	0.02662	0.02354	0.02310
31	Numbers of flasks used in measuring normal head-water samples		c.f.s.		0.02401	0.02394	0.02147	0.02545	0.02669	0.02624	0.02662	0.02352	0.02310
32	Average volume of flasks measuring normal head-water samples				4 & 5	6	6	1 456	1 474	1 474	2 10	1 476-2	2
33	Numbers of flasks used in measuring dosed tail-water samples at temperature of Line 4				1 474, 1 476	1 476, 1 474	1 476	1 456	1 474	1 474	1474-1476	1 476-2,	2
34	Average volume of flasks measuring dosed tail-water samples		c.c.		1 008.58	1 008.58	1 008.58	1 008.90	1 009.70	1 008.61	1 008.6	1 008.59	500.46
35	Density of Normal head-water when measured in laboratory at temperature of Line 5				9 & 8	11	11	11	3	11	3	500.44	11
36	Normal head-water at mean temperature of head and tail-water, temperature of Line 9				3 & 11							11	
37	Dosed tail-water when measured in laboratory at temperature of Line 4				500.51	500.60	500.61	500.61	500.45	500.69	500.62	500.66	500.65
38	Dosed tail-water at mean temperature of head and tail-water, temperature of Line 8		c.c.		0.9960	0.9963	0.9940	0.9977	0.9972	0.9963	0.9975	0.9973	0.9963
39	Volume of normal head-water samples at mean temperature of head and tail-water, temperature of Line 8				0.9960	0.9964	0.9964	0.9964	0.9964	0.9965	0.9968	0.9962	0.9965
40	Titration of normal head-water samples at mean temperature of head and tail-water, temperature of Line 8				0.9961	0.9963	0.9961	0.9961	0.9972	0.9965	0.9976	0.9973	0.9973
41	Titration per liter of normal head-water sample at mean temperature of head and tail-water, temperature of Line 8	Line 32 × Line 35 ÷ Line 36	c.c.		0.9965	0.9965	0.9965	0.9965	0.9965			0.9967	0.9967
42	Correction to titration of normal head-water per liter		c.c.		1 008.1	1 008.4	1 008.1	1 008.2	1 008.6	1 008.4	1 008.5	1 008.3—	499.56
43	Correct titration of normal head-water per liter		c.c.		25.365	25.350	25.379	25.440	25.388	25.389	25.383	499.79	
44	Volume of tail-water samples at mean temperature of head and tail-water, temperature of Line 8	Line 40 ÷ Line 39	c.c.		25.327	25.313	25.301	25.364	25.322	24.980	25.320	1 008.0	12.981
45	Titration of dosed tail-water samples at temperature of dosed tail-water when measured for evaporation, temperature of Line 5	Line 41 − Line 42	c.c.		−0.983	−0.988	−0.983	−0.983	−0.983	−0.988	−0.988	25.319	25.305
46	Titration per liter of dosed tail-water at mean temperature of head and tail-water	Line 34 × Line 37 ÷ Line 38	c.c.	v_1	25.034	25.040	25.036	25.096	25.050	24.735	25.037	−0.983	0.98
47	Approximate value of h	Line 43 × Line 44	c.c.		500.51	500.50	500.51	500.41	500.45	500.69	500.62	25.040	25.705
48	Value of $v = h v_1$	Line 44 × Line 37	c.c.		51.176	45.565	45.047	49.701	47.172	45.156	49.376	500.54	490.30
49	Value of $v_2 = v_1$		c.c.	v_2	109.329	91.037	91.035	99.479	94.486	90.293	97.569	51.545	55.205
50				h	0.9755	0.9756	0.9757	0.9753	0.9755	0.9762	0.9767	109.249	118.985
51	Value of r	Line 45 × Line 47		$hv_1 = kv_2$	98.322	88.807	88.807	97.032	92.188	88.520	93.522	0.9758	0.9744
52	Value of $Q_1 = qr$	Line 23 − Line 45		$v - kv_2$	548 960	547 461	547 361	554 923	555 475	539 142	539 784	100.036	111.009
53	Value of kq	Line 45 − Line 48		$v_2 - v_1$	77.335	65.997	60 217	74.335	69.427	65.496	62.615	548 720	564 059
54	Value of $Q_2 = Q_1 + kq$											77.902	89.280
		Line 49 ÷ Line 50	c.f.s.	r	7 099.99	6 295.94	6 268.17	7 463.91	7 299.55	6 216.16	6 416.82	7 042.97	6 692.79
		Line 30 × Line 51	c.f.s.	kq	1 661.5	1 661.9	1 667.4	1 902.6	1 647.1	1 776.7	1 767.7	1 640.0	1 716.1
		Line 30 × Line 47	c.f.s.	Q_2	0.023	0.199	0.197	0.230	0.201	0.211	0.235	0.027	0.206
		Line 52 + Line 53	c.f.s.		1 661.7	1 662.1	1 667.6	1 902.8	1 647.3	1 776.9	1 767.9	1 640.2	1 716.4*

←——Long Marks (1st Calibration) on Gauge Board of Salt Solution Tank Used on These Tests.——→

Note.—Missing Tests, *A, D, E*, represent rough preliminary tests by chemicals, and are not included in this table. All other chemical tests appear in the tables in the order in which they were executed; missing numbers, or letters, relate to tests by current meters or to tests for purposes other than turbine performance.

Missing Tests, *B, C, F, J, P, Q, R*, 1-5, incl., represent current-meter tests.

Missing Test, *M*, represents a distribution test.

No Leeds and Northrup or Allis-Chalmers power readings taken on lettered tests. (High-precision instruments.)

Some of the discharges are more or less questionable owing to the probability of leaks in the tail-race sampling pipes. In computing this table, titrations of doubtful samples have been omitted, figures being substituted instead, which are based on the titrations of adjacent samples. The table on the right gives the discharges of Tests 5, 10-13 incl., using the actual titrations, no alterations, or substitutions, being made. These are the only tests thus treated as being doubtful in this series.

*In Tests 7 and 9 the salt solution samples were undoubtedly interchanged by mistake in the laboratory. More correct values for the discharge in these tests can be computed by interchanging the values given above for v and recomputing Q_2. Thus, for Test 7, $Q_2 = 1\,668$, and for Test 9, $Q_2 = 1\,712$.

OR COMPUTATION OF TURBINE DISCHARGE. COMPILED FROM RECORDS OF TESTS MADE ON UNIT NO. 13, JULY 18TH, 1914, TO AUGUST 19TH, 1914.—

	Units.	Symbol.	TESTS. G	H	I	K	L	N	O	S	T	U	V.
.....	°C	.....	18.2	18.1	18.2	18.2	18.2	17.8	17.9	18.6	18.4	18.5	18.0
.....	°C	.....	21.0	20.0	21.0	24.0	23.0	22.5	22.0	26.0	24.0	20.0	22.5
.....	°C	.....	21.0	20.0	21.0	24.0	23.0	22.5	22.0	26.0	26.5	20.0	22.5
.....	°C	.....	21.0	20.0	21.0	23.0	23.5	20.0	22.5	25.0	25.0	25.0	19.0
.....	°C	.....	21.0	20.0	21.0	21.0	23.5	20.0	22.5	23.5	24.0	25.0	19.0
.....	°C	.....	19.4	19.3	19.4	19.4	19.0	19.0	15.6	18.3	18.6	18.3	18.5
.....	°C	.....	19.1	19.3	19.4	19.4	19.4	19.0	18.6	18.3	18.6	18.3	19.3
) ÷ 2	°C	.....	19.1	19.3	19.4	19.4	19.4	19.0	18.6	18.3	18.6	18.3	18.4
.....	.....	.....	No. 18 791	No. 18 791	No. 18 791	No. 18 791	No. 18 791	No. 18 791	No. 18 791	No. 18 791	No. 18 791	No. 18 791	No. 18 791
.....	c.c.	.....	9.980	9.979	9.980	9 953	9 984	9.982	9.982	9.985	9.983	9.980	9.982
.....	.....	.....	1 476	1 476	1 476	1 476	1 476	1 474, 1 476	1 476	1 456	1 456, 1 476	1 456, 1 476	1 456, 1 476
.....	c.c.	.....	1 002.87	1 003.50	1 003.52	1 003.60	1 003.63	1 003.60	1 003.55	1 003.00	1 003.52	1 003.50	1 003.52
.....	.....	.....	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047
.....	c.c.	.....	10.013	10.012	10.013	10.017	10.016	10.015	10.015	10.020	10.020	10.012	10.017
.....	c.c.	.....	54.556	54.504	54.479	55.185	55.229	53.512	52.605	54.572	56.103	53.425	54.490
14	c.c.	.....	5 458.50	5 445.90	5 440.80	5 509.10	5 513.00	5 348.90	5 252.90	5 446.40	5 600.60	5 337.10	5 439.80
10	.....	.....	100.49	100.56	100.55	100.53	100.52	100.54	100.54	100 45	100.50	100.55	100.50
17	c.c.	.....	548 520	547 440	547 070	553 830	554 170	537 210	528 130	547 090	562 700	536 050	546 700
.....	gms.	.....	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622
.....	gms.	.....	294.7	294.3	294.2	297.3	297.8	288.7	284.0	294.2	302.4	288.3	293.9
ves)	.....	.....	24.85	24.82	24.83	25.06	25.12	24.43	24.08	24.87	25.44	24.39	24.83
.....	.....	.....	1.1674	1.1672	1.1672	1.1893	1.1896	1.1644	1.1814	1.1674	1.1923	1.1835	1.1672
.....	.....	.....	1.1851	1.1854	1.1858	1.1902	1.1902	1.1830	1.1794	1.1827	1.1902	1.1827	1.1844
.....	.....	.....	1.1670	1.1806	1.1806	1.1888	1.1891	1.1830	1.1811	1.1675	1.1923	1.1833	1.1672
Line 23	c.c.	v	548 960	547 550	547 450	555 650	555 500	538 220	526 670	548 640	564 170	536 920	543 010
.....	.....	.....	8	8	29	6	43	10	13	13	11	9	8
.....	.....	.....	59	31	63	36	66	72	71	66	70	66	59
Line 25)	sec.	.....	1 519.9	972.4	1 009 0	1 127.0	946.1	2 422.6	2 320.1	1 518.0	1 529.0	1 571.1	1 500.8
Line 24	c.f.s.	q	0.23394	0.20094	0.20137	0.22536	0.20681	0.21613	0.20992	0.23284	0.27310	0.25790	0.22976
	c.f.s.	q'	0.23401	0.20084	0.20147	0.22545	0.20669	0.21624	0.20998	0.23282	0.27310	0.25795	0.22978
.....	.....	.....	4 & 8 / 1 474, 1 476	8 4 / 1 476, 1 474	1 476	1 456	1 474	1 474	2 10 / 1474-1476	1 476-8	8	11	11
.....	c.c.	.....	1 003.56	1 003.55	1 003.52	1 002.90	1 003.70	1 003.61	1 003.6	1 003.59	500.46	500.66	500.59
.....	.....	.....	9 & 5 / 2 & 11	11	11	11	3	11	3	11	11	11	11
.....	c.c.	.....	500.51	500.50	500.61	500.61	500.45	500.60	500.43	500.64	500.65	500.66	500.59
.....	.....	.....	0.9980	0.9983	0.9980	0.9977	0.9973	0.9983	0.9975	0.9973	0.9968	0.9983	0.9985
.....	.....	.....	0.9985	0.9984	0.9984	0.9984	0.9985	0.9985	0.9983	0.9983	0.9985	0.9985	0.9985
.....	.....	.....	0.9981	0.9983	0.9981	0.9981	0.9973	0.9983	0.9976	0.9973	0.9973	0.9985	0.9985
.....	.....	.....	0.9985	0.9985	0.9985	0.9985	0.9985	0.9985	0.9987	0.9987	0.9987	0.9985	0.9987
Line 26	c.c.	.....	1 003.1	1 003.4	1 003.1	1 002.3	1 002.6	1 003.4	1 002.5	1 003.2	499.56	499.81	500.54
.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	499.79	.....	.....	.....
.....	c.c.	.....	25.365	25.389	25.379	25.440	25.388	25.068	25.283	1 000.9 / 25.632	12.981	12.992	13.003
39	c.c.	.....	25.287	25.308	25.301	25.354	25.322	24.996	25.220	25.009	25.985	25.994	25.976
.....	c.c.	.....	—0.268	—0.268	—0.263	—0.263	—0.252	—0.262	—0.263	—0.263	0.28	0.28	0.28
42	c.c.	v_1	25.024	25.040	25.038	25.121	25.069	24.735	24.957	25.346	25.705	25.714	25.696
Line 38	c.c.	.....	500.51	500.50	500.41	500.41	499.65	500.50	499.88	499.94	499.90	499.76	500.49
.....	c.c.	.....	51.176	45.505	45.647	49.761	47.172	45.158	42.876	51.546	56.356	60.918	56.164
56	c.c.	v_2	102.299	91.057	91.255	99.479	94.486	90.333	87.869	103.243	113.025	121.880	110.289
.....	.....	k	0.9755	0.9756	0.9757	0.9753	0.9753	0.9762	0.9767	0.9753	0.9744	0.9762	0.9754
67	.....	kv_2	99.822	88.807	89.026	97.022	92.158	88.165	85.823	100.696	111.009	118.929	107.569
68	.....	$v - kv_2$	548 560	547 461	547 381	554 953	555 408	538 142	526 734	548 732	564 359	536 801	547 938
63	.....	$v_2 - v_1$	77.305	63.997	66 217	74.358	69.427	65.496	62.816	77.903	89.320	96.166	84.584
50	.....	r	7 099.92	8 295.94	8 266.17	7 463.26	7 999.58	8 216.16	8 418.58	7 042.97	6 299.73	5 569.02	6 477.61
51	c.f.s.	qr	1 661.5	1 661.9	1 665.4	1 682.6	1 647.1	1 776.7	1 767.2	1 640.0	1 716.1	1 429.9	1 488.2
47		$\frac{1}{2}q_2$	0.238	0.196	0.197	0.220	0.201	0.211	0.205	0.227	0.266	0.253	0.224
53	c.f.s.	Q_2	1 661.7	1 662.1	1 665.6	1 682.8	1 647.3	1 776.9	1 767.9	1 640.2	1 745.4*	1 430.3*	1 488.4

←——Long Marks (1st Calibration) on Gauge Board of Salt Solution Tank Used on These Tests.——→

←——Sampling Pipes on Lower

NOTE.—Missing Tests. *A, D, K*, represent rough preliminary tests by chemicals, and are not included in this table. All other chemical tests appear in the tables in the order in which they were executed; missing numbers, or letters, relate to tests by current meters or to tests for purposes other than turbine performance.

Missing Tests, *B, C, F, J, P, Q, R*, 1-5, incl., represent current-meter tests.

Missing Test, *M*, represents a distribution test.

No Leeds and Northrup or Allis-Chalmers power readings taken on lettered tests. (High-precision instruments.)

Some of the discharges are more or less questionable owing to the probability of leaks in the tail-race sampling pipes. In compiling this table, titrations of doubtful samples have been omitted, figures being substituted instead, which are based on the titrations of adjacent samples. The table on the right gives the discharge of Tests 8, 10-13 incl., using the actual titrations, no alterations, whatever, being made. These are the only tests thus treated as being doubtful, in this series.

*In Tests *T* and *U* the salt solution samples were undoubtedly interchanged by mistake in the laboratory. More correct values for the discharge in these tests can be computed by interchanging the values given above for v and recomputing Q_2. Thus, for Test *T*, $Q_2 = 1\,603$, and for Test *U*, $Q_2 = 1\,813$.

TABLE

Line.	
45.....	Titrat
46.....	Line
47.....	.....
48.....	Line
49.....	Line
50.....	Line
51.....	Line
52.....	Line
53.....	Line
54.....	Line

014.—SEE NOTE BELOW.

	W	6	7	8	9	10	11	12	13	14
	17.5 21.5 21.5 21.0 19.0 18.6 19.0 19.5 No. 18 791 9,981	18.0 22.0 22.0 22.0 21.5 19.0 19.5 19.25 No. 18 791 9,985	17.5 22.0 22.0 22.0 20.0 19.0 19.0 19.0 No. 18 791 9,981	17.5 22.0 22.0 22.0 20.0 19.0 19.0 19.0 No. 18 791 9,981	17.5 22.5 22.5 19.5 21.0 19.2 19.2 19.2 No. 18 791 9,983	17.5 24.0 24.0 22.0 22.0 19.1 19.0 19.0 No. 18 791 9,983	18.0 24.5 24.5 19.7 19.7 18.8 19.0 18.5 No. 18 791 9,984	17.9 23.0 23.0 22.0 21.0 18.7 19.0 18.5 No. 18 791 9,984	17.5 23.0 23.0 22.0 20.5 18.7 19.0 18.5 No. 18 791 9,983	17.5 23.0 23.0 22.0 20.5 18.7 19.0 18.5 No. 18 791 9,983
1 476 047 017 490 80 50 5683 9	1 456, 1 476 1 003.31 No. 21 047 10.014 53.376 5 330.30 100.51 555 750 1.5622 288.0	1 456, 1 476 1 003.30 No. 21 047 10.016 56.836 5 374.10 100.49 539 940 1.5622 284.8	1 456, 1 476 1 003.31 No. 21 047 10.015 54.653 5 454.00 100.51 543 160 1.5622 284.0	1 456, 1 476 1 003.32 No. 21 047 10.016 54.653 5 445.10 100.51 547 290 1.5622 284.0	1 456, 1 476 1 003.28 No. 21 047 10.017 54.653 5 396.10 100.50 542 610 1.5622 281.4	1 456, 1 476 1 003.27 No. 21 047 10.017 54.105 5 407.40 100.50 543 440 1.5622 281.8	1 456, 1 476 1 003.29 No. 21 047 10.018 53.443 5 330.30 100.49 535 120 1.5622 280.0	1 456, 1 476 1 003.30 No. 21 047 10.018 53.626 5 365.30 100.49 538 230 1.5622 280.0	1 456, 1 476 1 003.30 No. 21 047 10.015 50.841 5 073.00 100.47 509 290 1.5622 274.0	1 456, 1 476 1 003.30 No. 21 047 10.015 50.792 5 070.10 100.47 509 290 1.5622 273.8
83 1818 1844 1873 8 23976 23978	24.37 1.1839 1.1815 1.1835 556 460 65 60 1 231.8 0.24055 0.24067	24.16 1.1830 1.1795 1.1813 551 210 33 67 1 196.3 0.24051 0.24075	24.26 1.1828 1.1853 1.1871 548 890 5 25 1 208.0 0.21025 0.21088	24.52 1.1875 1.1853 1.1868 548 000 41 69 1 175.1 0.20178 0.20185	24.63 1.1862 1.1830 1.1856 543 640 9 55 1 658.5 0.23992 0.24011	24.63 1.1864 1.1830 1.1857 544 740 15 66 1 794.8 0.24056 0.24070	24.41 1.1839 1.1806 1.1835 537 470 10 61 1 632.9 0.23565 0.23573	25.29 1.1918 1.1876 1.1907 550 720 16 67 1 804.9 0.23230 0.23240	23.85 1.1758 1.1730 1.1751 511 230 8 57 1 089.4 0.23537 0.23551	23.86 1.1757 1.1710 1.1750 510 710 44 73 1 070.7 0.22140 0.22158
59 9965 9966 9965 9967 54 003	3 500.32 3 500.39 0.9965 0.9965 0.9965 0.9967 0.9966 500.32 13.730	11 500.63 11 500.62 0.9975 0.9964 0.9970 0.9965 0.9965 500.15 13.971	11 500.60 11 500.60 0.9963 0.9963 0.9963 0.9963 0.9963 500.50 13.050	11 500.60 11 500.60 0.9963 0.9963 0.9963 0.9963 0.9963 500.50 13.056	11 500.60 11 500.61 0.9964 0.9964 0.9961 0.9965 0.9965 500.60 13.056	23 500.35 23 500.33 0.9977 0.9965 0.9974 0.9974 0.9965 499.91 13.179	3 500.61 3 500.60 0.9954 0.9965 0.9964 0.9964 0.9966 500.35 13.278	11 500.68 11 500.61 0.9977 0.9965 0.9964 0.9951 0.9966 500.22 13.250	3 500.45 3 500.41 0.9977 0.9965 0.9965 0.9965 0.9966 500.03 13.226	3 500.41 3 500.41 0.9980 0.9965 0.9965 0.9965 0.9966 500.16 13.275
978 23 036 49 164 283 9754 569 584	23.440 0.23 25.150 500.34 84.094 108.124 0.9768 105.551 525 344 83.954	25.933 0.23 25.659 500.32 88.535 107.059 0.9765 104.576 531 105 81.406	25.074 0.23 25.704 500.50 80.054 100.026 0.9755 97.277 543 792 74.234	25.225 0.23 24.945 500.50 49.051 97.984 0.9755 95.382 547 904 73.059	23.101 0.23 23.231 500.61 87.650 115.251 0.9759 119.479 543 582 89.430	26.962 0.23 26.082 499.97 84.586 109.135 0.9759 109.494 544 684 78.068	26.337 0.23 25.337 500.30 81.496 108.948 0.9759 108.498 537 270 78.021	25.829 0.23 25.549 500.26 81.615 109.206 0.9749 109.007 550 689 77.959	25.874 0.23 25.294 500.51 83.428 108.883 0.9779 104.422 511 116 80.559	25.543 0.23 25.262 500.51 83.451 104.920 0.9779 102.931 510 607 78.056
61 2 234 4	6 464.73 1 555.9 0.235 1 836.1	6 594.15 1 570.7 0.235 1 570.9	7 229.73 1 555.2 0.235 1 556.6	7 501.53 1 514.9 0.197 *1 514.4	6 077.69 1 459.5 0.234 1 459.5	6 557.67 1 570.4 0.235 *1 572.6	7 009.95 1 561.7 0.220 *1 561.9	7 237.04 1 728.4 0.235 *1 725.6	6 244.69 1 483.5 0.234 *1 481.0	6 421.45 1 483.1 0.217 1 483.9

*Note.—See table below.

Short Marks (3d Calibration) on Gauge Board of Salt Solution Tank Used on These Tests.

Lower Section.

TABLE GIVING DISCHARGE OF TESTS 8, 10-13, INCL., USING ACTUAL DOSED TAIL-WATER TITRATIONS.

	Formula.	Symbol.	Tests.				
			8	10	11	12	13
	Titration of D. T. W.	.....	63.836	54.323	51.329	51.429	53.836
	Line 45 × Line 38	v_2	97.668	108.729	103.587	102.961	107.274
	Line 44 × Line 37	k	0.9755	0.9758	0.9768	0.9248	0.9779
	.....	$k\,v_2$	95.245	106.093	100.027	100.295	104.903
	Line 35 − Line 48	$v - kv_2$	547 905	544 624	552 570	559 600	511 115
	Line 46 − Line 45	$v_2 - v_1$	72.402	82.647	76.260	77.413	80.999
	Line 49 ÷ Line 50	r	7 557.39	6 590.62	7 044.70	7 230.08	6 311.52
	Line 30 × Line 51	$Q_1 = qr$	1 521.39	1 556.18	1 660.25	1 730.97	1 488.39
	Line 30 × Line 47	kq	0.197	0.235	0.220	0.235	0.221
	Line 52 + Line 53	Q_2	1 521.6	1 556.4	1 660.5	1 731.1	1 488.55

Number of line in this table corresponds to number of line in main table.

Test 202, September 30th, 1914.

Titration, October 2d.
Nitrate (1.56 grammes), September 9th.

Dilute salt solution. Temp. 19° cent. Flasks Nos. 1 456–1 474.		Normal head-water. Temp. 16.5° cent. Flasks Nos. 3–11.	Special dilutions.* Temp. 16.5° cent. Flasks Nos. 1 456–		
5 050	5 050	1 295	5 000	5 065	10 065
5 052	5 058	1 289	4 995	5 053	10 048
5 033	5 040	1 283	5 035	5 087	10 122
5 070	5 070	1 283	5 085	5 055	10 140
			5 060	5 015	10 075
Mean = 50.529		Mean = 12.875	Mean = 50.450		

Tail-Water Titrations.
Temp. 16.5° cent. Flasks Nos. 3-11.

5 235	5 185	5 185	5 205	5 170	5 145
5 250		5 265		5 310	
	5 468		5 800		5 794
5 582		†5 811		5 933	
	5 724		5 910		5 962

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper.....	52 240	52 075	52.167
Lower.....	56 462	58 798	57.760
Averages.	54.351	55.437	54.963 = Arith. mean.

Weighted mean = 54.564

* 10-c.c. pipettes of salt solution from each of Samples ———, 30 c.c., diluted with 2 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

† Average of two values to the right and the lower one to the left. Sample pipe was broken.

Test 203, October 1st, 1914.

Titration, October 3d.
Nitrate (1.56 grammes). September 9th.

Dilute salt solution. Temp. 17° cent. Flasks Nos 1 456–1 476.		Normal head-water. Temp. 15° cent. Flask No. 11.	Special dilutions.* Temp. 15° cent. Flasks Nos. 1 456–1 476.		
5 110	5 115	1 270	5 038	5 053	10 091
5 115	5 120	1 282	5 035	5 060	10 095
5 112	5 118	1 280	5 040	5 050	10 090
5 136	5 140	1 280	5 020	5 073	10 093
			5 041	5 056	10 097
Mean = 51.208		Mean = 12.780	Mean = 50.466		

Tail-Water Titrations.
Temp. 15.4° cent. Flask No. 11.

4 910	4 925	4 880	4 865	4 785	4 945
4 965		4 915		4 938	
	5 327		5 961		6 315
5 460		†5 905		6 343	
	5 668		6 085		6 376

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper.....	49 190	48 842	49 031
Lower.....	55 900	62 160	59 378
Averages.	52.545	55.501	54.204 = Arith. mean.

Weighted mean = 53.646

* 30 c.c. salt solution from a mixture of Samples *A*, *C*, *E*, *G*, *D*, *K*, 30 c.c., diluted with 2 000 c.c. normal head-water and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

† Average of the two values on the right and the lower one on the left. Sample pipe was broken.

TEST 204, OCTOBER 2d, 1914.

TITRATION, OCTOBER 4TH.
Nitrate (1.56 grammes), September 9th.

Dilute salt solution. Temp. 17.75° cent. Flasks Nos. 1 456–1 474.		Normal head-water. Temp. 16.8° cent. Flask No. 11.	Special dilutions.* Temp. 16.8° cent. Flasks Nos. 1 456–1 474.		
4 721	4 724	1 260	4 759	4 706	9 465
4 723	4 723	1 250	4 755	4 713	9 468
4 715	4 715	1 251	4 764	4 715	9 479
4 720	4 715	1 250	4 760	4 700	9 460
			4 750	4 710	9 460
Mean = 47.195		Mean = 12.528	Mean = 47.332		

TAIL-WATER TITRATIONS.
Temp. 16° cent. Flask No. 11.

4 581		4 609		4 580	
	4 607		4 625		4 652
4 650		4 630		4 719	
	5 105		5 633		5 825
5 145		†5 608		5 913	
	5 370		5 822		5 967

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper.....	46 154	46 440	46.281
Lower.....	53 070	58 320	55.987
Averages.	49.612	52.380	51.134 = Arith. mean.

Weighted mean = 50.351.

*30 c.c. salt solution from a mixture of samples *A*, *C*, *E*, *G*, *D*, *K*, 30 c.c., diluted with 2 000 c.c. normal head water, and 10 c.c. of this diluted to 1 000 c. c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

†Average of two values on the right and the lower one on the left. Sample pipe broken.

TEST 205, OCTOBER 5th, 1914.

TITRATION OCTOBER 6TH.
Nitrate (1.56 grammes), September 9th.

Dilute salt solution. Temp. 23.5° cent. Flasks Nos. 1 456–1 474.		Normal head-water. Temp. 17° cent. Flask No. 11.	Special dilutions.* Temp. 17° cent. Flasks Nos. 1 456–1 474.		
4 653	4 644	1 240	5 082	5 043	10 125
4 642	4 644	1 251	5 065	5 053	10 118
4 635	4 645	1 255	5 072	5 030	10 102
4 645	4 645	1 239	5 050	5 035	10 085
			5 050	5 035	10 085
Mean = 46.441		Mean = 12.463	Mean = 50.515		

TAIL-WATER TITRATIONS.
Temp. 16° cent. Flasks Nos. 3–11.

5 647		5 703		5 450	
	5 687		5 560		5 430
5 710		5 635		5 507	
	5 255		4 000		3 982
4 753		3 895		3 755	
	4 214		3 680		3 750

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper.....	56 764	54 868	55.921
Lower.....	45 292	38 334	41.427
Averages.	51.028	46.601	48.674 =Arith. mean.

Weighted mean = 49.797.

*10 c.c. salt solution from each of Samples *B*, *D*, *F*, *I*, *K*, 50 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

TABLE 51.—Data for Computation of Turbine Discharge. Compiled from Records of Tests Made on Unit N

Line.	UNIT No. 12.	Formula.	Symbol.	Units.	15	16	17	18	19	20	21	22 A	
1	Temperature: Salt solution in tank during test			°C	17.6	17.6	17.7	17.7	17.65	17.4	17.4	17.7	
2	Temperature: Salt solution when pipetted for dilution with distilled water			°C	25.0	25.0	25.5	25.5	25.5	17.0	17.0	24.0	
3	Temperature: Distilled water for dilution of salt solution			°C	25.0	25.0	25.5	25.5	25.5	17.0	17.0	24.0	
4	Temperature: Normal head-water when measured for evaporation in laboratory			°C	21.75	21.75	23.0	23.0	21.0	23.0	21.0	25.25	
5	Temperature: Dosed tail-water " " " " " "			°C	21.75	21.75	23.0	23.0	21.0	21.75	24.0	30.5	
6	Temperature: Normal head-water in canal			°C	19.0	18.8	18.8	18.8	18.8	18.7	18.75	19.0	
7	Temperature: Dosed tail-water in tail-race			°C	19.0	18.8	18.8	18.8	18.8	19.0	.....	19.0	
8	Mean temperature of head and tail-water during test, mean of Line 6 and Line 7			°C	18.9	18.9	18.65	18.65	18.8	18.85	18.75	19.0	
9	Number of pipette used in measuring out salt solution for dilute salt solution at temperature of Line 2			.....	No. 18 791	No. 18 791	No. 18 791	No. 18 791	No. 18 791	No. 18 791	No. 18 791	No. 18 791	N
10	Volume of salt solution discharged by pipette for dilute salt solution at temperature of Line 2			c.c.	9.985	9.985	9.980	9.980	9.983	9.977	9.977	9.984	
11	Numbers of flasks used in measuring distilled water			.....	Nos. 1 456, 1 476	Nos. 1 456, 1 476	Nos. 1 456, 1 476	Nos. 1 456, 1 476	Nos. 1 456, 1 476	Nos. 1 456, 1 476	Nos. 1 456, 1 476	Nos. 1 456, 1 476	Nos.
12	Volume of flask measuring distilled water for dilution of salt solution at temperature of Line 3			c.c.	1 003.3	1 003.3	1 003.18	1 003.18	1 003.36	1 003.1	1 003.1	1 003.27	
13	Number of pipette used in measuring out dilute salt solution			.....	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	N
14	Volume of dilute salt solution discharged by pipette at temperature of Line 3			c.c.	10.018	10.018	10.018	10.018	10.016	10.006	10.006	10.017	
15	Titration per pipette of dilute salt solution in laboratory			c.c.	53.155	53.39	54.108	54.075	53.154	53.026	53.15	53.107	
16	Titration per liter of dilute salt solution in laboratory at temperature of Line 3	Line 15 ÷ Line 14		c.c.	5 305.9	5 330.4	5 400.5	5 400.5	5 305.4	5 299.3	5 305.8	5 301.7	
17	Ratio of dilution of salt solution with distilled water	Line 12 ÷ Line 10			100.48	100.48	100.52	100.52	100.51	100.54	100.54	100.49	
18	Titration per liter of salt solution at temperature of Line 3	Line 16 × Line 17		c.c.	532 140	534 880	542 320	542 320	533 200	532 750	532 750	532 720	53
19	Concentration of silver nitrate, grammes per liter (approximate)			gms.	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	
20	Concentration (approximate) in grammes per liter of salt solution at temperature of Line 3			gms.	296.5	297.0	291.8	291.7	303.6	296.3	296.9	296.3	
21	Percentage of salt in solution (approximate) at temperature of Line 3	$p = \frac{c}{d}$ (from curves)			24.3	24.33	24.64	24.64	25.44	24.31	24.36	24.33	
22	Density: Salt solution in tank during test at temperature of Line 1				1.1835	1.1835	1.1800	1.1800	1.1693	1.1835	1.1830	1.1830	
23	Density: " " when pipetted for dilution with distilled water at temperature of Line 3				1.1795	1.1797	1.1844	1.1844	1.1895	1.1825	1.1832	1.1799	
24	Density: " " at mean temperature of head and tail-water at temperature of Line 8		s		1.1327	1.1330	1.1355	1.1355	1.1223	1.1330	1.1324	1.1322	
25	Titration of salt solution per liter at mean temperature of head and tail-water at temperature of Line 8	Line 18 × Line 24 ÷ Line 22	q	c.c.	534 390	536 030	543 320	543 320	564 800	532 430	533 280	533 540	53
26	Tank gauge board reading at start			div.	9	43	9	15	5	62	62	15	
27	" " " finish			div.	37	69	65	65	65	27	70	69	
28	Time, in seconds, from first to last gauge reading			sec.	1 104.0	1 069.8	1 082.6	1 120.1	1 795.1	1 104.4	1 069.4	1 729.7	
29	Actual rate of discharge of salt solution from tank during test at temperature of Line 1	0.466 × (Line 27 − Line 26) ÷ Line 28		cu. ft.	0.21472	0.21267	0.21114	0.21163	0.24534	0.22231	0.22165	0.23544	
30	Equivalent rate of discharging salt solution at mean temperature of head and tail-water, temperature of Line 8	Line 29 × Line 22 ÷ Line 24	Q	cu. ft.	0.21483	0.21275	0.21123	0.21171	0.24536	0.22242	0.22177	0.23562	
31	Numbers of flasks used in measuring normal head-water samples			.....	11	12	3	3	1 456	3	3	11	
32	Average volume of flasks measuring normal head-water samples at temperature of Line 6			c.c.	500.63	500.62	500.45	500.44	1 002.87	500.44	500.41	500.60	
33	Numbers of flasks used in measuring dosed tail-water samples			.....	11	12	3	3	11	11	11	11	
34	Average volume of flasks measuring dosed tail-water samples, at temperature of Line 5			c.c.	500.62	500.62	500.44	500.44	500.61	500.62	500.65	500.60	
35	Density: Normal head-water when measured in laboratory, at temperature of Line 4				0.9978	0.9978	0.9968	0.9974	0.9980	0.9974	0.9980	0.9982	
36	Density: Normal head-water at mean temperature of head and tail-water, temperature of Line 6				0.9985	0.9985	0.9985	0.9985	0.9985	0.9985	0.9985	0.9985	
37	Density: Dosed tail-water when measured in laboratory at temperature of Line 5				0.9972	0.9972	0.9976	0.9975	0.9980	0.9976	0.9972	0.9982	
38	Density: Dosed tail-water at mean temperature of head and tail-water, temperature of Line 6				0.9985	0.9985	0.9985	0.9985	0.9985	0.9985	0.9985	0.9985	
39	Volume of normal head-water samples at mean temperature of head and tail-water, temperature of Line 6	Line 32 × Line 35 ÷ Line 36		c.c.	500.27	500.27	499.61	499.59	1 002.27	499.59	500.15	500.45	
40	Titration of normal head-water samples in laboratory			c.c.	12.923	13.010	13.000	13.000	25.404	13.005	13.045	13.126	
41	Titration per liter of normal head-water samples at mean temperature of head and tail-water, temperature of Line 6	Line 40 ÷ Line 39		c.c.	25.826	26.006	26.020	26.006	25.344	26.120	26.242	26.234	
42	Correction to titration of normal head-water per liter	Line 41 − Line 42	v_1	c.c.	0.29	0.29	0.29	0.29	0.29	0.29	0.29	0.29	
43	Correct titration per liter of normal head-water at mean temperature of head and tail-water, temperature of Line 6			c.c.	25.536	25.716	25.740	25.726	25.054	25.910	25.952	25.952	
44	Volume of dosed tail-water samples at mean temperature of head and tail-water, temperature of Line 6	Line 34 × Line 37 ÷ Line 38		c.c.	500.27	500.27	499.59	499.59	500.31	500.27	499.95	500.45	
45	Titration of dosed tail-water samples at temperature of tail-water when measured for evaporation, temperature of Line 5			c.c.	49.864	50.007	47.627	48.599	39.508	50.399	50.001	58.570	
46	Titration per liter of dosed tail-water samples at mean temperature of head and tail-water, temperature of Line 6	Line 45 × Line 38 ÷ Line 44 × Line 37	v_2	c.c.	97.744	100.380	95.519	97.205	117.043	100.811	100.188	117.071	
47	Approximate value of k		k	.....	0.9765	0.9764	0.9759	0.9759	0.9747	0.9764	0.9764	0.9765	
48	Value of $k\,v_2$	Line 45 × Line 47	$k\,v_2$	.....	97.410	98.450	93.217	94.892	114.053	98.433	97.799	114.320	
49	Value of $v - k\,v_2$	Line 42 − Line 48	$v - k\,v_2$	.....	534 493	535 593	543 527	543 255	564 686	532 332	533 282	533 726	53
50	Value of $v_2 - v_1$	Line 46 − Line 43	$v_2 - v_1$	.....	74.198	73.104	69.779	71.479	91.949	74.201	74.790	91.119	
51	Value of r	Line 49 ÷ Line 50	r	.....	7 204.05	7 195.98	7 789.28	7 600.39	6 141.39	7 167.01	7 130.39	5 897.45	
52	Value of $Q_1 = q\,r$	Line 30 × Line 51	Q_1	.....	1 547.6	1 535.5	1 643.2	1 630.0	1 506.5	1 580.7	1 581.3	1 497.3	
53	Value of $k\,q$	Line 30 × Line 47	$k\,q$	.....	0.210	0.209	0.205	0.207	0.239	0.217	0.217	0.250	
54	Value of $Q_2 = Q_1 + k\,q$	Line 52 + Line 53	Q_2	.....	* 1 547.8	1 535.7	* 1 643.4	* 1 630.2	* 1 507.0	* 1 580.9	* 1 581.5	1 497.5	
												Ave. =	

← Sampling pipes on lower section and also eight samples on upper section in Tests 19, 20, and 22. →

Note.—See Table 52 for Tests 22.B and 22.B * See table below for discharge. † Test 22 retitrated as Test 22.B 19 days after test of turbine

Table Giving Discharge of Tests 15, 17-21, Incl., Using Actual Dosed Tail-Water Titrations.

Line.	Formula.	Symbol.	Tests. 15	17	18	19	20	21
45	Titration D. T. W.	.....	49.864	47.627	47.954	58.193	50.079	49.819
46	Line 45 × Line 38	v_2	99.760	95.520	95.085	116.354	100.162	99.736
47	.....	k	0.9765	0.9759	0.9759	0.9747	0.9764	0.9764
48	Line 46 × Line 47	$k\,v_2$	97.467	93.223	92.781	113.429	97.796	97.423
49	Line 42 − Line 48	$v - k\,v_2$	534 426	543 527	543 256	564 697	532 332	533 282
50	Line 46 − Line 43	$v_2 - v_1$	73.941	69.880	70.309	91.320	74.253	74.426
51	Line 49 ÷ Line 50	r	7 226.30	7 811.54	7 726.09	6 186.51	7 169.13	7 165.93
52	Line 30 × Line 51	$Q_1 = q\,r$	1 553.00	1 649.95	1 633.82	1 517.91	1 594.56	1 589.04
53	Line 30 × Line 47	$k\,q$	0.210	0.205	0.207	0.239	0.217	0.217
54	Line 52 + Line 53	Q_2	1 553.2	1 650.2	1 634.0	1 517.4	1 594.8	1 589.3

Number of line in this table corresponds to number of line in main table.

A FOR COMPUTATION OF TURBINE DISCHARGE. COMPILED FROM RECORDS OF TESTS MADE ON UNIT NO. 13, AUGUST 19TH TO SEPTEMBER 2D, 1914.—SEE NOTE

							TEST.					
15	16	17	18	19	20	21	22 *A*	22 *B*	23 *A*	26 *A*	32 *A*	32 *B*
17.6	17.6	17.7	17.7	17.65	17.4	17.4	17.7	17.7	16.8	16.5	16.95	16.95
25.0	25.0	20.5	20.5	23.5	17.0	17.0	24.0	17.0	21.0	19.5	19.5	17.5
25.0	25.0	20.5	20.5	23.5	17.0	17.0	24.0	17.0	21.0	19.5	19.5	17.5
21.75	21.75	23.0	23.0	21.0	21.0	21.0	30.25	18.95	21.0	19.0	19.0	19.0
21.75	21.75	23.0	23.0	21.0	21.75	24.0	20.5	18.0	21.0	19.5	19.0	19.0
18.8	18.8	18.8	18.8	18.8	18.7	19.75	19.0	19.0	18.8	18.5	18.7	18.7
19.0	19.0	18.5	18.5	18.8	19.0	.....	19.0	19.0	18.8	18.25	.....	.....
18.9	18.9	18.65	18.65	18.8	18.85	18.75	19.0	19.0	18.8	18.4	18.7	18.7
No. 18 791	No. 18 791	No. 18 791	No. 18 791	No. 18 791	No. 18 791	No. 18 791	No. 18 791	No. 18 791	No. 18 791	No. 18 791	No. 18 791	No. 18 791
9.985	9.965	9.960	9.980	9.968	9.977	9.977	9.984	9.977	9.960	9.960	9.960	9.975
Nos. 1 456, 1 476	Nos. 1 456, 1 476	Nos. 1 456, 1 476	Nos. 1 456, 1 476	Nos. 1 456, 1 476	Nos. 1 456, 1 476	Nos. 1 456, 1 476	Nos. 1 456, 1 476	Nos. 1 456, 1 476	Nos. 1 456, 1 476	Nos. 1 456, 1 476	Nos. 1 456, 1 476	Nos. 1 456, 1
1 003.2	1 003.3	1 003.16	1 003.16	1 003.36	1 003.1	1 003.1	1 003.27	1 003.1	1 003.2	1 003.16	1 003.17	1 003.11
No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047
10.018	10.018	10.018	10.013	10.016	10.008	10.008	10.017	10.008	10.013	10.013	10.013	10.009
53.155	53.30	54.102	54.075	56.154	53.035	53.13	55.107	53.361	54.437	52.858	52.537	53.422
5 230.4	5 402.3	5 402.5	5 402.5	5 406.4	5 299.3	5 306.6	5 301.7	5 331.8	5 435.6	5 279.5	5 247.4	5 239.6
100.48	100.48	100.56	100.56	100.51	100.54	100.54	100.49	100.54	100.52	100.56	100.56	100.53
534 590	543 130	542 860	542 860	563 500	532 790	533 750	532 770	536 000	546 490	530 700	527 470	526 640
1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	1.562
296.5	297.0	291.6	291.7	303.6	286.3	286.9	286.3	288.1	293.4	285.3	283.3	283.0
24.3	24.33	24.64	24.64	25.44	24.21	24.26	24.33	24.35	24.75	24.17	24.00	23.98
1.1833	1.1835	1.1860	1.1866	1.1926	1.1826	1.1830	1.1830	1.1836	1.1874	1.1826	1.1813	1.182
1.1795	1.1797	1.1844	1.1844	1.1895	1.1808	1.1808	1.1796	1.1830	1.1858	1.1812	1.1796	1.180
1.1827	1.1829	1.1855	1.1855	1.1922	1.1820	1.1824	1.1822	1.1830	1.1865	1.1816	1.1803	1.180
536 080	536 080	543 630	543 330	564 800	532 490	533 880	533 840	535 630	547 040	530 360	527 680	526 490
9	43	40	40	16	9	42	15	13	12	12	14	14
57	59	58	58	58	57	70	69	69	63	63	66	66
1 104.0	1 069.8	1 062.6	1 130.1	1 795.1	1 104.4	1 069.4	1 789.7	1 789.7	1 791.9	1 784.8	1 801.2	1 801.2
0.21472	0.21367	0.21114	0.21163	0.34594	0.22231	0.22166	0.35544	0.35544	0.34195	0.34195	0.33971	0.339
0.21483	0.21378	0.21122	0.21171	0.34536	0.22249	0.22177	0.35568	0.35557	0.34194	0.34214	0.33990	0.339
11	11	3	3	1 456	3	3	11	11	11	11	3	11
500.62	500.62	500.46	500.44	1 002.37	500.44	500.41	500.60	500.58	500.61	500.37	500.39	500.37
11	11	3	3	11	11	11	11	11	11	11	3	11
500.62	500.62	500.44	500.44	500.61	500.62	500.55	500.59	500.57	500.61	500.53	500.39	500.57
0.9976	0.9976	0.9968	0.9974	0.9990	0.9974	0.9980	0.9982	0.9987	0.9990	0.9987	0.9995	0.998
0.9985	0.9985	0.9965	0.9985	0.9985	0.9985	0.9985	0.9985	0.9985	0.9986	0.9986	0.9985	0.998
0.9976	0.9976	0.9975	0.9975	0.9990	0.9976	0.9978	0.9982	0.9987	0.9990	0.9986	0.9995	0.998
0.9985	0.9985	0.9986	0.9986	0.9986	0.9985	0.9986	0.9985	0.9985	0.9986	0.9986	0.9985	0.998
500.57	500.57	499.81	499.89	1 002.37	499.69	500.16	500.45	500.68	500.81	500.62	500.39	500.57
13.033	13.010	13.000	13.000	25.404	13.002	13.035	13.126	13.368	13.971	13.006	13.016	13.033
25.533	26.006	26.020	26.006	25.344	26.190	25.642	26.232	26.700	25.926	25.974	26.012	26.031
0.33	0.33	0.26	0.26	0.33	0.25	0.33	0.25	0.25	0.25	0.25	0.25	0.25
25.562	25.725	25.740	25.725	25.004	25.910	25.262	25.952	26.420	25.646	25.694	25.728	25.781
500.57	500.57	499.89	499.89	500.31	500.57	499.95	500.45	500.67	500.81	500.56	500.54	500.62
49.564	50.407	47.597	45.539	58.506	50.398	50.001	58.570	59.096	55.940	56.376	52.776	53.536
99.744	100.830	95.519	97.205	117.013	100.811	100.168	117.071	118.010	117.877	112.433	107.499	106.993
0.9766	0.9764	0.9739	0.9739	0.9747	0.9764	0.9764	0.9765	0.9763	0.9755	0.9765	0.9769	0.976
97.410	98.490	93.017	94.763	114.053	98.438	97.788	114.330	115.213	114.969	109.780	104.995	104.448
583 032	583 032	543 527	543 255	564 695	532 232	533 252	533 736	535 515	546 995	530 750	527 575	526 579
74.122	75.104	69.779	71.479	91.949	74.901	74.790	91.119	91.590	92.231	86.726	81.757	81.173
7 354.05	7 185.96	7 789.26	7 600.20	6 141.30	7 107.01	7 130.32	5 807.46	5 846.97	5 929.95	6 119.71	6 488.96	6 494.22
1 547.5	1 535.5	1 645.8	1 609.0	1 508.8	1 580.7	1 561.2	1 497.3	1 494.3	1 490.6	1 491.5	1 546.1	1 550.1
0.210	0.209	0.205	0.207	0.239	0.217	0.217	0.250	0.250	0.235	0.236	0.234	0.234
1 547.5	1 535.7	* 1 645.4	* 1 609.2	* 1 507.0	* 1 580.9	* 1 561.5	1 497.5	†1 494.5	1 490.7	1 492.0	1 546.3	‡1 555.3
							Ave. =	1 496.0			Ave. =	1 551.8

—Sampling pipes on lower section and also eight samples on upper section in Tests 19, 22, and 23.—>

<—Short marks (second calibration) on gauge-board of salt solution

Table 53 for Tests 22*B* and 32*B* * See table below for discharge. † Test 22 retitrated as Test 22*B* 19 days after test of turbine. ‡ Test 32 retitrated as Test 32*B* 15 days after the turbine test. There is,

Missing Tests 2
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doubtful samples h
the left gives the di

ING DISCHARGE OF TESTS 15, 17–21, INCL., USING ACTUAL DOSED TAIL-WATER TITRATIONS.

	Symbol.	Tests.					
		15	17	18	19	20	21
W.	.....	49.688	47.597	47.964	58.193	50.073	49.819
38	v_2	99.398	95.220	96.085	116.384	100.168	99.788
37	k	0.9766	0.9739	0.9739	0.9747	0.9764	0.9764
47	$k\,v_2$	97.067	93.022	93.571	113.429	97.796	97.433
46	$v - k\,v_2$	534 493	543 527	543 256	564 687	532 232	533 253
45	$v_2 - v_1$	73.841	69.580	70.309	91.320	74.252	74.425
39	r	7 236.30	7 811.54	7 726.69	6 188.61	7 109.13	7 185.36
41	$Q_1 = q\,r$	1 535.03	1 649.95	1 635.52	1 517.21	1 504.56	1 559.04
42	$k\,q$	0.210	0.206	0.207	0.239	0.217	0.217
43	Q_2	1 555.3	1 650.2	1 636.0	1 517.4	1 504.5	1 559.3

Number of line in this table corresponds to number of line in main table.

NOTE BELOW.

38 B	38	39	40	41	42	43	44	45 A	45 B	46
16.95	16.8	16.5	17.33	17.3	17.53	17.7	18.37	18.1	19.1	18.83
17.5	19.8	22.5	21.0	24.5	22.0	20.0	22.0	22.0	17.0	19.5
17.5	19.5	22.5	21.0	24.5	22.0	20.0	22.0	22.0	17.0	19.5
18.0	21.75	19.0	22.0	20.0	21.5	22.5	23.0	25.3	17.0	21.5
18.0	21.75	19.0	20.5	20.0	20.5	22.5	22.0	24.0	17.0	21.5
18.7	18.9	16.7	19.0	18.8	19.0	19.5	19.25	20.5	20.5	20.5
.....	18.65	18.5	19.0	18.9	19.0	19.2	20.00	19.9	19.9	19.8
18.7	18.7	18.6	19.0	18.95	19.0	19.25	19.62	20.2	20.2	20.15
o. 16 791	No. 16 791	No. 16 791	No. 16 791	No. 16 791	No. 16 791	No. 16 791	No. 16 791	No. 16 791	No. 16 791	No. 16 791
9.978	9.980	9.968	9.981	9.968	9.988	9.988	9.987	9.981	9.976	9.983
1 456, 1 476	Nos. 1 456, 1 476	Nos. 1 456, 1 476	Nos. 1 456, 1 476	Nos. 1 456, 1 476	Nos. 1 456, 1 476	Nos. 1 456, 1 476	Nos. 1 456, 1 476	Nos. 1 456, 1 476	Nos. 1 456, 1 476	Nos. 1 456, 1 476
1 008.11	1 008.37	1 008.37	1 008.3	1 008.25	1 008.4	1 008.32	1 008.37	1 008.33	1 008.1	1 008.16
o. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047	No. 21 047
10.009	10.012	10.017	10.013	10.018	10.022	10.020	10.009	10.015	10.008	10.012
53.429	53.429	53.351	53.412	51.471	54.611	54.246	54.429	54.367	54.526	51.327
5 398.6	5 395.9	5 395.9	5 394.3	5 197.9	5 449.6	5 413.6	5 439.37	5 446.5	5 449.4	5 196.5
100.53	100.52	100.50	100.51	100.46	100.46	100.46	100.47	100.51	100.55	100.53
3 640	535 420	641 220	535 150	516 260	547 270	543 990	543 670	547 630	547 940	515 220
1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622
293.0	293.9	290.5	297.7	277.9	294.0	293.1	293.1	294.0	294.3	277.9
23.98	24.37	24.60	24.35	23.61	24.68	24.73	24.80	24.80	24.75	23.55
1.1806	1.1843	1.1862	1.1837	1.1778	1.1889	1.1865	1.1871	1.1871	1.1899	1.1797
1.1806	1.1826	1.1822	1.1822	1.1743	1.1819	1.1824	1.1822	1.1822	1.1875	1.1794
1.1803	1.1833	1.1852	1.1830	1.1772	1.1873	1.1857	1.1864	1.1861	1.1856	1.1759
5 480	535 630	542 630	535 580	517 590	549 290	545 500	547 890	548 070	547 170	515 170
14	17	16	13	11	32	13	24	13	25	25
65	68	68	62	70	58	56	67	65	58	69
1 801.2	1 785.4	1 817.7	1 794.1	1 795.6	1 473.7	1 505.6	1 510.3	1 509.7	1 509.7	1 504.9
0.23971	0.24188	0.24219	0.24201	0.24073	0.24144	0.24124	0.24104	0.24113	0.24113	0.24192
0.23969	0.24202	0.24236	0.24216	0.24065	0.24153	0.24151	0.24118	0.24132	0.24135	0.24207
11	11	11	11	11	11	11	11	11	11. 8	11
500.57	500.62	500.59	500.62	500.60	500.62	500.66	500.62	500.67	500.45	500.62
11	11	11	11	11	11	11	11	11	11. 8	11
500.57	500.62	500.59	500.60	500.60	500.60	500.64	500.62	500.65	500.45	500.62
0.9987	0.9978	0.9965	0.9977	0.9983	0.9978	0.9965	0.9974	0.9987	0.9966	0.9978
0.9985	0.9985	0.9985	0.9985	0.9985	0.9985	0.9984	0.9984	0.9983	0.9983	0.9983
0.9987	0.9978	0.9985	0.9982	0.9988	0.9982	0.9973	0.9978	0.9972	0.9989	0.9979
0.9986	0.9986	0.9986	0.9985	0.9986	0.9985	0.9985	0.9984	0.9983	0.9983	0.9982
500.57	500.57	500.59	500.58	500.59	500.57	499.70	500.13	500.63	500.77	500.57
13.038	13.067	13.071	13.174	13.166	13.155	13.105	13.974	13.944	13.896	13.817
26.051	25.840	26.111	26.346	26.349	26.315	26.223	25.941	25.895	28.754	25.615
0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26
25.791	25.580	25.851	26.086	26.083	26.052	25.943	25.681	25.635	28.474	25.355
500.62	500.52	500.54	500.43	500.43	500.54	500.54	500.52	500.10	500.76	500.42
53.536	55.399	55.905	55.402	54.069	56.127	55.736	55.958	54.549	55.007	55.227
106.929	112.626	113.825	110.738	108.061	112.127	111.697	113.809	109.196	§119.974	110.405
0.9759	0.9759	0.9759	0.9759	0.9775	0.9755	0.9759	0.9753	0.9759	0.9759	0.9760
104.443	109.957	111.070	108.114	103.630	109.425	109.005	111.143	106.564	110.321	107.976
6 376	536 520	542 519	535 479	517 444	549 791	545 291	547 779	547 953	547 060	515 062
81.173	87.076	87.994	84.652	81.995	86.155	85.754	86.236	85.551	86.500	85.070
6 484.24	6 161.27	6 163.41	6 222.23	6 210.62	6 251.30	6 259.95	6 227.97	6 550.07	6 224.89	6 054.57
1 553.1	1 491.2	1 494.6	1 531.2	1 519.9	1 541.6	1 535.0	1 497.3	1 585.1	1 585.4	1 462.6
0.324	0.236	0.237	0.295	0.295	0.295	0.295	0.235	0.236	0.236	0.237
1 555.3	1 491.4	1 494.6	1 531.5	1 520.1	1 541.8	1 530.3	1 497.4	1 582.3	§1 224.6	1 465.8
1 551.8										

Sampling pipes on upper section.

olution tank used on these tests.

There is, therefore, some doubt as to the accuracy of 38 *B*. It would be better, perhaps, to disregard the *B* tests entirely.

Tests 34, 35, 37-41, incl., and 44-53, incl., represent current-meter tests. Some of the discharges, Tests 15-21, incl., are more stionable, owing to the probability of leaks in tail-race sampling pipes. In compiling the above table, titrations of mples have been omitted, figures being substituted, which are based on the titrations of adjacent samples. The table on es the discharge of Tests 15, 17-21, incl., using the actual titrations, no alterations, whatever, being made.

§ The titrations of tail-water samples in Test 45 *B* were made about a week after Test 45. They indicate that the samples had become contaminated, one of them being 72.45 c.c., as against about 54 c.c., the correct value. This reduction of Test 45, consequently, is disregarded.

TEST 206, OCTOBER 6TH, 1914.

TITRATION, OCTOBER 7TH.
Nitrate (1.56 grammes), October 5th.

Dilute salt solution. Temp. 18° cent. Flasks Nos. 1 456-1 476.		Normal head-water. Temp. 13.8° cent. Flask No. 11.	Special dilutions.* Temp. 13.8° cent. Flasks Nos. 1456-1476.		
4 550	4 560	1 253	4 965	4 960	9 925
4 560	4 552	1 262	5 005	4 970	9 975
4 561	4 552	1 255	4 985	4 983	9 968
4 558	4 559	1 256	4 990	4 978	9 968
			4 948	5 018	9 966
Mean = 45.565		Mean = 12.565	Mean = 49.802		

TAIL-WATER TITRATIONS.
Temp. 14° cent. Flasks Nos. 3-11.

5 385	5 340	5 330	5 146	5 043	4 863
5 405		5 233		5 030	
4 580	5 247	4 460	4 370	4 504	4 510
	4 525		4 488		4 478

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper	53 386	50 205	51.972
Lower	47 030	44 700	45.786
Averages.	50.208	47.452	48.854 = Arith. mean.

Weighted mean = 49.480

*10 c.c. salt solution from each of Samples *B*, *D*, *F*, *I*, *K*, 50 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

TEST 207, OCTOBER 7TH, 1914.

TITRATION, OCTOBER 8TH.
Nitrate (1.56 grammes), October 5th.

Dilute salt solution. Temp. 18.5° cent. Flasks Nos. 1 456-1 476.		Normal head-water. Temp. 12.75° cent. Flask No. 11.	Special dilutions.* Temp. 12.75° cent. Flasks Nos. 1 456-1 476		
4 684	4 679	1 253	5 119	5 032	10 151
4 682	4 680	1 250	5 107	5 053	10 160
4 680	4 680	1 252	5 119	5 040	10 159
4 675	4 680	1 255	5 080	5 065	10 145
			5 075	5 070	10 145
Mean = 46.800		Mean = 12.525	Mean = 50.760		

TAIL-WATER TITRATIONS.
Temp. 14° cent. Flasks Nos. 3-11.

5 515	5 500	5 447	5 275	5 075	4 950
5 560		5 325		5 197	
4 620	5 330	4 690	4 718	4 651	4 749
	4 650		4 850		4 726

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper	54 694	51 242	53.160
Lower	48 225	47 388	47.760
Averages.	51.460	49.315	50.460 = Arith. mean.

Weighted mean = 50.685

*10 c.c. salt solution from each of Samples *B*, *D*, *F*, *I*, *X*. 50 c.c., diluted with 3 000 c.c. normal head water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

TEST 208, OCTOBER 8TH, 1914.

TITRATION, OCTOBER 9TH.
Nitrate (1.56 grammes), October 5th.

Dilute salt solution. Temp. 18.25° cent. Flasks Nos. 1 456–1 476.		Normal head-water. Temp. 16.7° cent. Flask No. 11.	Special dilutions.* Temp. 18° cent. Flasks Nos. 1 456–1 476.		
4 830	4 840	1 250	5 250	5 202	10 452
4 835	4 830	1 258	5 312	5 210	10 522
4 845	4 838	1 260	5 255	5 205	10 460
4 838	4 837	1 261	5 245	5 180	10 425
			5 210	5 200	10 410
Mean = 48.366		Mean = 12.572	Mean = 52.269		

TAIL-WATER TITRATIONS.
Temp. 16.25° cent. Flask No. 11.

5 750		5 708		5 163	
	5 655		5 432		5 025
5 722		5 520		5 200	
	5 509		4 585		4 540
4 800		4 615		4 569	
	4 670		4 556		4 501

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper.....	56 710	52 200	54.706
Lower.....	48 985	45 502	47.050
Averages.	52.848	48.851	50.878 = Arith. mean.

Weighted mean = 51.753

*10 c.c. salt solution from each of Samples *B*, *D*, *F*, *I*, *K*, 50 c.c., diluted with 3 000 c.c. normal head water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts. 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

TEST 209, OCTOBER 9TH, 1914.

TITRATION, OCTOBER 10TH.
Nitrate (1.56 grammes), October 5th.

Dilute salt solution. Temp. 21° cent. Flasks Nos. 1 456–1 476.		Normal head-water. Temp. 16° cent. Flasks Nos. 3–11.	Special dilutions.* Temp. 16° cent. Flasks Nos. 1 456–1 476.		
4 990	4 990	1 252	5 370	5 295	10 665
4 987	4 990	1 264	5 375	5 295	10 670
4 980	4 990	1 252	5 350	5 345	10 695
5 002	5 003	1 260	5 370	5 305	10 675
			5 360	5 315	10 675
Mean = 49.915		Mean = 12.570	Mean = 53.380		

TAIL-WATER TITRATIONS.
Temp. 16° cent. Flasks Nos. 3–11.

5 752		5 800		5 020	
	5 602		5 365		4 820
5 615		5 450		5 206	
	5 225		4 772		4 542
4 850		4 782		4 550	
	4 815		4 610		4 485

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper.....	56 438	51 028	54.033
Lower.....	49 180	45 918	47.368
Averages.	52.809	48.473	50.701 = Arith. mean.

Weighted mean = 51.477

*10 c.c. salt solution from each of Samples *B*, *D*, *F*, *I*, *K*, 50 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

TABLE 32.—Data for Computation of Turbine Discharge. Compiled from Records of Tests Made on Units Nos. 11, 12, and 13. Tests

2.—Data for Computation of Turbine Discharge. Compiled from Records of Tests Made on Units Nos. 11, 12, and 13. Tests 101–104, inclusive, Ar

Formula.	Symbol.	Units.	UNIT No. 11.—Dosed Tabled Sampling Pipes on Upper Section. Sept. 15 to Sept. 18, 1914. This Unit Continued on Table 83.					UNIT No. 12.—Dosed Ta					
			105	106	107	108	109	200	201	202	203	204	
		°C	14.6	15.	16.1	15.8	16.0	15.4	12.4	12.4	12.3	12.23	
		°C	15.0	22.0	19.5	23.0	20.0	15.0	13.5	13.0	17.0	17.75	
		°C	16.0	23.0	19.5	23.0	20.0	15.0	15.0	14.5	16.5	16.2	
		°C	16.0	22.0	19.5	22.0	20.0	15.75	15.5	15.0	17.0	17.75	
		°C	15.25	21.0	21.0	22.5	20.0	15.0	15.0	14.5	15.0	16.5	
		°C	15.25	19.5	21.0	21.0	20.0	15.0	14.5	14.5	14.5	16.3	
		°C	15.25	21.0	20.5	19.0	23.5	15.5	15.3	16.5	15.4	16.9	
		°C	17.0	17.1	17.5	18.0	18.0	16.1	16.0	16.0	15.5	15.52	
		°C	17.0	17.3	17.6	17.7	18.0	16.1	16.0	15.5	15.5	15.3	
$\frac{1}{2}$ (+ Line 8) ÷ 2		°C	17.0	17.2	17.6	17.8	18.0	16.05	16.1	15.8	15.5	15.3	
		c.c.	No. 18 791 9.978 Nos. 1 456, 1 474 1 008.19 No. 21 047 10.010	No. 18 791 9.995 Nos. 1 456, 1 474 1 008.39 No. 21 047 10.014	No. 18 791 9.990 Nos. 1 456, 1 474 1 008.23 No. 21 047 10.018	No. 18 791 4 × 9.982 No. 1 456, 1 474 3 × 1 008.32 No. 21 047 10.016	No. 18 791 4 × 9.982 Nos. 1 456, 1 474 3 × 1 008.84 No. 21 047 10.015	No. 18 791 9.976 Nos. 1 456 1 008.74 No. 21 047 10.007	No. 18 791 9.976 Nos. 1 456, 1 476 1 008.06 No. 21 047 10.017	No. 18 791 9.976 Nos. 1 456, 1 476 1 008.28 No. 21 047 10.011	No. 18 791 9.992 Nos. 1 456, 1 476 1 008.10 No. 21 047 10.010	No. 18 791 9.990 Nos. 1 456, 1 476 1 008.16 No. 21 047 10.010	No
		c.c.											
		c.c.											
		c.c.	58.514	42.850	29.755	56.926	61.224	46.800	51.469	50.539	64.936	47.195	
17 + Line 16		c.c.	5 846.05	4 295.94	2 971.92	5 681.81	6 121.05	4 676.7	5 142.5	5 047.5	6 481.82	4 714.6	
14 × Line 13		c.c.	100.84	100.67	100.43	99.898	75.571	100.50	100.25	100.32	100.63	100.69	
15 × Line 19		c.c.	587 493	434 939	298 441	423 504	461 350	470 010	517 109	507 350	514 330	473 930	
			1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	1.5622	
			288.58	288.58	160.43	227.7	240.0	288.58	288.58	288.58	288.58	288.58	
d (from curve)			64.40	20.22	14.51	19.90	21.01	21.75	23.54	23.50	23.50	21.50	
			1.1554	1.1523	1.1051	1.1463	1.1908	1.1523	1.1706	1.1707	1.1706	1.1523	
			1.1536	1.1490	1.1049	1.1448	1.1895	1.1534	1.1705	1.1740	1.1771	1.1535	
			1.1538	1.1490	1.1049	1.1449	1.1895	1.1534	1.1704	1.1705	1.1705	1.1545	
			1.1543	1.1515	1.1055	1.1475	1.1894	1.1530	1.1705	1.1705	1.1720	1.1543	
Line 24 + Line 25		c.c.	292 940	436 190	296 709	424 317	462 093	469 950	517 320	505 920	513 000	474 740	
Line 27 ÷ Line 26	g	c.c.	587 530	435 930	296 540	424 520	461 710	469 870	517 009	505 110	514 740	474 850	
		Div.	59	18	20	15	14	21	85	41	59	59	
		Div.	44	45	44	49	45	64	73	73	73	73	
Line 31 — Line 30)		sec.	840.6	1 237.4	1 300.8	1 158.1	1 190.4	1 505.0	1 795.4	1 781.9	1 805.9	1 781.3	
Line 33		cu. ft.	0.24171	0.24138	0.24791	0.24068	0.24180	0.24128	0.24128	0.24171	0.24135	0.24039	
Line 34 ÷ Line 27	q	cu. ft.	0.24195	0.24155	0.24800	0.24090	0.24199	0.24138	0.24205	0.24190	0.24202	0.24061	
		c.c.	9.976	9.992	9.990	9.982	9.982	9.976	9.976	9.977	9.976	9.979	
Line 36 ÷ Line 37		c.c.	9.975	9.994	9.995	9.984	9.976	9.981	9.974	9.974	9.977	9.976	
		c.c.	1 456, 1 474 $Y = 3\ 000\ m = 4\ 3\ 008.8$	1 456, 1 474 $Y = 3\ 000\ m = 4\ 3\ 008.8$	1 456, 1 474 $Y = 3\ 000\ m = 4\ 3\ 008.8$	1 456, 1 474 $Y = 3\ 000\ m = 4\ 3\ 008.9$	1 456, 1 474 $Y = 3\ 000\ m = 4\ 3\ 008.9$	1 456, 1 $Y = 3\ 000\ m = 4\ 3\ 008.9$	1 456 $Y = 3\ 100\ m = 3\ 2\ 994.1$	1 456 $Y = 2\ 900\ m = 2\ 3\ 008.5$	1 456, 1 476 $Y = 3\ 000\ m = 3\ 2\ 994.1$	1 456, 1 474 $Y = 3\ 000\ m = 3\ 3\ 008.8$	1 $Y =$
		c.c.	0.9995	0.9980	0.996	0.9975	0.9983	0.9991	0.9991	0.9989	0.9991	0.9989	
			0.9986	0.9987	0.9980	0.9985	0.9983	0.9991	0.9990	0.9989	0.9991	0.9989	
			0.9986	0.9986	0.9987	0.9987	0.9987	0.9990	0.9990	0.9991	0.9991	0.9991	
Line 40 ÷ Line 42		c.c.	3 009.0	3 007.4	3 007.7	3 008.9	3 008.8	3 008.8	3 008.8	3 008.7	3 008.8	3 008.8	
+ m × Line 26	r_0		75.414	75.407	75.305	75.420	75.401	75.356	79.327	67.016	67.984	67.094	
	R_0		0.978	0.982	0.980	0.983	0.981	0.982	0.977	0.978	0.977	0.980	
64 + Line 65	R_0		78.380	78.420	79.294	78.412	78.382	78.356	81.304	68.994	69.961	69.094	
		c.c.	10.010	10.013	10.013	10.013	10.015	10.005	10.008	10.005	10.008	10.008	
			1.0018	1.0008	1.0008	0.9996	1.0006	1.0018	1.0020	1.0019	1.0022	1.0017	
			1.0016	1.0010	1.0003	1.0010	1.0010	1.0015	1.0012	1.0012	1.0013	1.0019	
Line 43 ÷ Line 42		c.c.	10.007	10.006	10.006	10.003	10.008	10.007	10.008	10.008	10.008	10.005	
		c.c.	1 008.2	1 008.36	1 008.38	1 008.3	1 008.34	1 008.75	1 008.72	1 008.75	1 008.05	1 008.15	
Line 40 ÷ Line 42		c.c.	1 008.0	1 008.5	1 008.2	1 008.1	1 008.8	1 008.6	1 008.3	1 008.4	1 008.0	1 008.9	
Line 50 ÷ Line 50	R_0		100.58	100.19	100.30	100.13	100.10	100.51	100.41	100.44	100.54	100.53	
53 × Line 46	R'		7 656.57	7 656.02	7 644.96	7 654.95	7 652.42	7 649.58	7 644.96	6 619.90	6 616.49	6 618.04	
	h'		0.976	0.982	0.980	0.983	0.981	0.980	0.977	0.978	0.977	0.980	
$R' - K'$	h'		7 655.18	7 657.44	7 645.07	7 652.09	7 652.09	7 648.24	7 643.26	6 618.22	6 615.84	6 616.43	
	h		0.976	0.982	0.980	0.983	0.981	0.980	0.977	0.977	0.977	0.980	
			1 456, 1 474	1 456, 1 474	1 456, 1 474	1 456, 1 474	1 456, 1 474	1 456, 11	1 456	1 456	1 456, 1 476	1 456, 1 474	1
		c.c.	501.03	501.54	501.54	501.55	501.56	501.36	501.36	501.35	501.53	501.55	
		c.c.	48.055	41.440	32.390	40.305	43.478	48.509	49.135	50.490	50.466	47.333	
60 ÷ Line 59		c.c.	98.809	98.909	94.169	81.843	86.675	86.783	98.005	100.029	100.024	94.366	
			0.9987	0.9981	0.9981	0.9976	0.9982	0.9983	0.9981	0.9990	0.9993	0.9981	
			0.9989	0.9993	0.9993	0.9993	0.9987	0.9991	0.9991	0.9992	0.9992	0.9989	
Line 63 ÷ Line 62	v_2	c.c.	95.522	98.975	94.018	81.841	86.710	86.705	97.963	100.043	100.014	94.375	
			3, 11	3, 11	3, 11	3, 11	3, 11	11	11	3, 11	11	3, 11	
		c.c.	500.48	500.48	500.51	500.50	500.50	500.54	500.52	500.48	500.54	500.55	
			11	3	3	11	11	5, 11	2	5, 11	11	11	
		c.c.	500.56	500.41	500.49	500.59	500.52	500.48	500.34	500.48	500.54	500.55	
			0.9987	0.9991	0.9992	0.9993	0.9976	0.9992	0.9993	0.9990	0.9992	0.9991	
			0.9989	0.9989	0.9993	0.9993	0.9987	0.9991	0.9991	0.9992	0.9992	0.9993	
Line 41 ÷ Line 42		c.c.	500.28	500.42	500.16	500.49	500.49	500.59	500.52	500.40	500.54	500.45	
		c.c.	13.125	12.998	13.098	12.956	13.100	13.077	12.950	12.875	13.190	13.093	
74 ÷ Line 73		c.c.	25.250	25.564	26.053	25.861	25.170	25.257	25.257	25.179	25.179	25.253	
		c.c.	0.29	0.29	0.29	0.29	0.29	0.29	0.29	0.29	0.29	0.29	
75 — Line 76	v_1	c.c.	25.950	25.744	25.845	25.611	25.899	25.943	25.957	25.449	25.959	25.783	
Line 71 + Line 73		c.c.	500.43	500.01	500.10	500.49	500.08	500.54	500.52	500.54	500.54	500.55	
		c.c.	97.990	97.057	92.309	80.980	85.778	85.762	96.740	98.954	98.946	93.591	
79 ÷ Line 68		c.c.	115.700	94.007	88.349	97.441	101.155	97.539	105.208	109.029	107.176	100.561	
Line 78 ÷ Line 71	v_2	c.c.	115.729	94.172	88.401	97.460	101.202	97.535	105.184	109.059	107.176	100.591	
81 — Line 84	$v_2 - v'_2$		+ 19.931	+ 11.497	+ 21.769	+ 16.519	+ 14.556	+ 11.051	+ 10.251	+ 6.410	+ 6.564	+ 8.332	
85 × Line 51	$k' v_2$		119.958	93.477	85.587	95.803	99.343	95.859	105.090	106.544	104.711	98.659	
Line 85	$v - k' v_2$		427 540	435 790	296 504	424 494	461 930	469 774	516 894	508 032	514 935	474 951	
84 × Line 82			+ 138 875	+ 66 048.9	+ 105 501	+ 121 494	+ 111 406	+ 84 514.5	+ 73 804.7	+ 57 920.0	+ 64 728.8	+ 42 [illegible].7	
86 + Line 84			0.93346	0.90304	0.85799	0.88833	0.84123	+ 0.19019	+ 0.14100	+ 0.11279	+ 0.06491	+ 0.08943	
Line 87 + 1	$1 + f$		1.93346	1.90304	1.85799	1.88831	1.84123	1.19019	1.14100	1.11279	1.06491	1.08943	
88 ÷ Line 88	v		5 964.1	4 370.57	4 908.11	5 904.95	6 164.76	6 483.49	6 893.89	6 921.75	6 870.61	6 [illegible].26	
84 × Line 89	Q		1 412.19	1 026.5	1 511.30	1 497.5	1 491.6	1 384.33	1 313.46	1 480.65	1 317.58	1 026.00	
e as Line 87	k		0.976	0.982	0.980	0.982	0.981	0.980	0.977	0.977	0.977	0.980	
94 × Line 91	$k q$		0.95207	0.95873	0.96466	0.94555	0.92739	0.95690	0.95645	0.95584	0.95649	0.95260	
90 ÷ Line 93	Q_2		1 483.4	1 539.0	+ 1 518.9	1 497.7	1 495.0	1 564.5	1 515.7	1 481.1	1 517.9	1 505.3	
as Line 87	k'		0.976	0.982	0.980	0.982	0.981	0.980	0.977	0.977	0.977	0.980	
94 × Line 81	$k v_2$		119.958	93.477	85.587	95.803	99.343	95.859	105.090	106.544	104.711	98.659	
89 — Line 95	$v - k v_2$		307 540	435 790	296 504	424 494	461 930	469 774	516 894	508 032	514 935	474 951	
81 — Line 77	$v_2 - v_1$		89.770	68.428	60.553	71.849	75.267	72.183	79.293	63.027	63.008	70.548	
96 ÷ Line 97	r		5 967.37	6 388.59	4 929.62	5 907.17	6 124.95	6 508.10	6 508.03	6 978.97	6 871.59	6 [illegible].54	
98 ÷ Line 84	Q_1		1 465.6	1 033.5	1 518.09	1 475.8	1 482.2	1 392.74	1 314.75	1 489.67	1 320.54	1 [illegible].43	
94 × Line 34	$k q$		0.236	0.237	0.235	0.2	0.227	0.237	0.236	0.236	0.236	0.236	
99 + Line 100	Q_2		1 448.8	1 538.5	1 518.9	1 475.8	1 482.4	1 571.0	1 515.0	1 470.1	1 520.6	1 504.7	
93 + Line 101			1 448.1	1 538.8	1 518.9	1 491.8	1 497.3	1 567.8	1 515.4	1 478.6	1 519.3	1 505.0	

Line 84, Test 107) appears to be in error. $R' = \frac{v - k' v_1}{v_2 - v_1}$. R' by formula = 7 701.90.

DOSED TAIL SAMPLING PIPES ON UPPER SECTION. SEPT. 23 TO OCT. 13, 1914.

UNIT No. 12.
SEE TABLES 50 AND 51.

205	206	207	208	209	210	211	212	25 B	25 D
13.3 13.5 17.0 (21.5) 17.0 17.0 14.0 14.5 14.0 14.25	13.1 13.0 13.5 17.25 13.5 13.75 13.5 14.0 13.1 13.25 13.5	13.5 13.5 13.75 13.5 13.75 13.75 14.0 14.0 13.75 14.0 13.37	13.0 13.0 13.0 13.0 13.0 13.0 14.0 14.0 14.0 14.0 14.0	13.0 13.0 13.0 13.0 14.0 13.5 17.5 17.5 14.05 14.1 14.1	13.5 14.0 13.5 17.5 17.5 17.5 14.0 14.0 14.4 14.0 14.5	14.0 13.25 13.0 13.0 13.5 13.5 14.0 14.0 14.0 14.0 14.0	11.9 13.5 11.0 13.0 11.5 11.0 13.0 13.0 14.1 13.25	13.5 (20) 13.5 (13.0) 13.5 14.5 14.5 13.5 13.5 13.5	13.5 13.25 13.5 13.25 13.0 13.5 13.5 13.25 13.5 13.4
No. 18 791 9,957 Nos. 1 458, 1 474 1 008.10 No. 21 047 10.016 45.645 4 585.7 100.46 465 930	No. 18 791 9,961 Nos. 1 458, 1 476 1 008.10 No. 21 047 10.020 45.660 4 588.4 100.50 45 930	No. 18 791 9,961 Nos. 1 458, 1 476 1 008.10 No. 21 047 10.014 45.638 4 579.9 100.44 465 720	No. 18 791 9,950 Nos. 1 458, 1 476 1 008.10 No. 21 047 10.011 45.624 4 581.9 100.42 465 640	No. 18 791 9,952 Nos. 1 458, 1 476 1 008.10 No. 21 047 10.015 45.642 4 584.0 100.50 500 800	No. 18 791 9,950 Nos. 1 458, 1 476 1 008.10 No. 21 047 10.013 45.631 4 819.5 100.32 484 490	No. 18 791 9,950 Nos. 1 458, 1 476 1 008.15 No. 21 047 10.011 45.640 4 588.7 100.38 458 400	No. 18 791 9,918 Nos. 1 458, 1 476 1 008.15 No. 21 047 10.011 45.776 4 575.2 100.54 458 880	No. 18 791 9,956 Nos. 1 458, 1 474 1 008.15 No. 21 047 10.012 45.644 5 430.5 100.40 545 760	No. 18 791 9,978 Nos. 1 458, 1 474 1 008.15 No. 21 047 10.012 45.638 5 202.5 100.34 520 940
1.5522 250.39 21.03 1.1555 1.1505 1.1515 1.1513 1.1504 457 760 457 920 16	1.5522 249.94 21.25 1.1551 1.1501 1.1513 1.1502 1.1503 458 480 458 640 20	1.5522 252.10 21.72 1.1551 1.1515 1.1513 1.1542 1.1543 471 050 470 700 14	1.5522 251.05 22.03 1.1701 1.1573 1.1575 1.1705 1.1688 485 850 485 810 17	1.5522 253.05 22.00 1.1750 1.1710 1.1735 1.1743 503 240 503 240 27	1.5522 250.44 22.54 1.1556 1.1571 1.1573 1.1553 1.1553 485 810 485 280 18	1.5522 252.52 22.50 1.1701 1.1693 1.1705 1.1702 1.1701 458 280 458 280 15	1.5522 253.32 22.52 1.1717 1.1693 1.1732 1.1702 1.1710 458 880 458 880 15	1.5522 252.32 24.26 1.1720 1.1697 1.1695 1.1701 1.1719 547 760 547 810 19	1.5522 253.32 24.12 1.1720 1.1697 1.1695 1.1701 1.1719 551 520 550 350 19
97 1 794.5 0.54105 0.54259 9.980 9.968 1 454, 1 474 V = 3 000 m = 5 3 009.5 0.9999	71 1 799.8 0.54198 0.54198 9.977 9.976 1 455, 1 476 V = 3 000 m = 5 3 009.1 0.9993	72 2 000.6 0.54181 0.54198 9.976 9.979 1 455, 1 476 V = 3 000 m = 5 3 009.0 0.9994	55 1 507.4 0.54180 0.54197 9.959 9.964 1 456, 1 476 V = 3 000 m = 5 3 009.4 0.9999	70 1 501.0 0.54228 0.54270 9.977 9.971 1 455, 1 476 V = 3 000 m = 5 3 009.2 0.9999	58 1 503.2 0.54207 0.54234 9.980 9.967 1 455, 1 476 V = 3 000 m = 5 3 009.2 0.9997	57 1 472.2 0.54182 0.54198 9.978 9.977 1 455, 1 476 V = 3 000 m = 5 3 009.1 0.9993	70 1 515.5 0.54249 0.54264 9.978 9.968 1 455, 1 476 V = 3 000 m = 5 3 009.9 0.9996	68 1 781.9 0.54105 0.54154 9.955 9.966 1 455 V = 3 000 m = 4 3 009.8 0.9999	62 1 794.5 0.54105 0.54214 9.975 9.970 1 455 V = 3 000 m = 4 3 009.67 0.9998
0.9993 0.9993 3 009.3 60.555 0.981 61.534 10.020 1.0015 1.0023 10.006	0.9993 0.9994 3 009.9 60.567 0.981 61.309 10.015 1.0028 1.0023 10.004	0.9994 0.9994 3 009.9 60.567 0.980 61.327 10.015 1.0028 1.0023 10.004	0.9992 0.9995 3 007.9 60.557 0.979 61.336 10.010 1.0017 1.0024 10.004	0.9992 0.9995 3 008.1 60.561 0.979 61.319 10.010 1.0022 1.0025 10.004	0.9997 0.9995 3 007.8 60.555 0.979 61.334 10.010 1.0015 1.0024 10.005	0.9993 0.9993 3 009.1 60.565 0.979 61.302 10.015 1.0025 1.0024 10.004	0.9993 0.9995 3 009.8 60.605 0.979 61.334 10.020 1.0025 1.0025 10.004	0.9973 0.9995 3 004.1 (2.494 0.979 76.410 10.014 1.0022 1.0015 9.990	0.9993 0.9995 200.80 .... 50.160 10.015 1.0022 1.0023 10.010
1 008.16 1 008.9 100.52 6 147.45 0.980 6 146.52 0.980 1 455, 1 474 501.50 50.015	1 008.05 1 008.9 100.55 6 146.45 0.979 6 144.55 0.979 1 455, 1 476 501.51 49.999	1 008.39 1 008.9 100.55 6 146.45 0.979 6 142.65 0.979 1 455, 1 476 501.50 50.760	1 008.19 1 008.4 100.51 6 144.45 0.977 6 142.55 0.979 1 456, 1 476 501.55 49.992	1 008.07 1 008.2 100.54 6 145.58 0.977 6 143.84 0.977 1 455, 1 476 501.54 49.990	1 008.11 1 008.6 100.51 6 144.71 0.979 6 143.73 0.979 1 455, 1 476 501.56 51.906	1 008.09 1 008.9 100.54 6 144.77 0.978 6 143.79 0.978 1 455, 1 476 501.51 52.323	1 008.55 1 008.9 100.59 6 145.67 0.978 6 145.19 0.978 1 455, 1 476 501.49 52.355	1 008.0 1 001.4 100.15 7 021.49 0.976 7 021.49 0.977 1 456 501.49 49.545	1 008.9 1 008.5 100.59 6 025.73 0.976 6 023.73 0.976 1 455, 1 474 501.49 50.782
100.719 0.9999 0.9993 100.752 11 500.56 11.0 500.45	99.304 0.9994 0.9995 99.344 11 500.56 11.0 500.59	101.316 0.9995 0.9995 101.310 11 500.51 11.0 500.55	104.313 0.9997 0.9994 104.309 11 500.55 11.0 500.55	104.428 0.9991 0.9994 104.464 11 500.45 11.0 500.51	103.469 0.9995 0.9993 103.511 11 500.57 11.0 500.57	104.382 0.9994 0.9994 104.327 9 500.59 9 500.56	104.491 0.9997 0.9995 104.527 9 500.59 9 500.51	95.599 0.9998 0.9995 97.575 5.11 502.55 11 500.55	103.139 0.9997 0.9995 103.119 5.11 502.47 11 500.59
0.9991 0.9995 500.55 12.455 24.909 0.99 24.908 500.59 49.819 24.904	0.9994 0.9995 500.51 12.565 25.105 0.99 25.099 500.59 49.959 25.079	0.9994 0.9995 500.51 12.565 25.084 0.99 24.744 500.59 49.965 101.333	0.9991 0.9994 500.55 12.573 25.125 0.99 24.546 500.60 51.703 103.321	0.9991 0.9994 500.59 12.570 25.105 0.99 24.708 500.59 51.677 103.261	0.9997 0.9995 500.59 12.545 25.095 0.99 24.794 500.59 50.664 101.213	0.9991 0.9994 500.59 12.545 25.074 0.99 24.794 500.59 50.650 101.201	0.9995 0.9995 500.44 12.565 25.078 0.99 24.790 500.51 50.574 104.425	0.9970 0.9995 499.90 12.550 27.925 0.99 27.925 498.90 50.650 119.706	0.9995 0.9995 500.59 12.553 25.457 0.99 25.457 500.59 50.743 115.523
59.594 — 1.525 57.594 457 552 — 7 345.14 — 0.01515 0.99995 6 947.42 1 519.00	59.555 — 0.429 59.126 458 240 — 2 026.42 — 0.00375 0.99995 6 180.09 1 491.18	101.333 + 0.977 102.310 470 651 + 672.127 + 0.00101 0.99995 6 127.43 1 455.14	103.422 — 0.893 101.521 485 450 — 5 304.41 — 0.01091 0.99999 6 813.29 1 501.57	103.296 — 2.272 100.025 503 199 — 21 955.7 — 0.04272 1.00003 6 426.51 1 550.74	101.264 — 0.277 99.987 484 121 — 19 026.1 — 0.03899 0.97115 6 827.05 1 539.19	101.201 — 0.000 99.201 458 121 — 19 026.9 — 0.03899 0.97111 6 868.32 1 548.98	104.425 + 0.102 104.194 458 121 + 635.036 + 0.00129 1.00109 6 197.57 1 469.15	119.706 + 22.661 117.039 547 195 + 174 345 + 0.21971 1.21971 6 797.55 1 526.97	115.523 — 19.720 95.802 520 720 — 99 526.9 — 0.19715 0.99995 6 182.56 1 497.13
0.980 0.99718 1 512.9 1 492 97.594 457 552 74.550 6 948.99 1 510.73 0.997	0.980 0.99545 1 491.6 1 492 98.907 458 240 74.550 6 185.69 1 491.09 0.996	0.979 0.99559 1 491.6 1 492 99.103 470 651 74.549 6 148.49 1 497.81 0.997	0.979 0.99718 1 501.5 1 492 101.521 485 450 74.577 6 190.51 1 495.90 0.997	0.977 0.99718 1 550.9 1 492 100.025 503 199 73.047 6 426.29 1 551.48 0.997	0.979 0.99543 1 539.4 1 492 99.192 484 121 73.472 6 829.61 1 542.98 0.996	0.978 0.99541 1 546.1 1 492 99.004 458 121 73.497 6 868.79 1 548.98 0.996	0.978 0.99720 1 469.4 1 492 100.194 458 121 73.535 6 191.57 1 469.69 0.997	0.979 0.99545 1 526.9 1 492 117.039 547 195 73.551 6 798.00 1 491.58 0.996	0.979 0.99542 1 497.4 1 492 119.023 555 720 67.195 6 206.55 1 472.55 0.996
1 511.0 1 511.0	1 498.5 1 498.4	1 495.0 1 498.7	1 498.2 1 499.0	1 551.7 1 550.8	1 541.9 1 541.9	1 545.9 1 545.0	1 492.9 1 491.9	*2 491.8 1 490.4	*2 474.1 1 488.8

* Test 25 reiltrated as Test 25 D one month after the test and 25 D 16 days after the test. Tests 25 B and 25 D are disregarded in the final computations owing to the probability of changes in samples when stored for so long a time.

TEST 210, OCTOBER 10TH, 1914.

TITRATION, OCTOBER 12TH.
Nitrate (1.56 grammes), October 5th.

Dilute salt solution. Temp. 18° cent. Flasks Nos. 1 456-1 476.		Normal head-water. Temp. 17.6° cent. Flask No. 11.	Special dilutions.* Temp. 17.6° cent. Flasks Nos. 1 456-1 476.		
4 834	4 823	1 257	5 195	5 186	10 381
4 827	4 820	1 250	5 166	5 197	10 363
4 825	4 830	1 260	5 197	5 200	10 397
4 825	4 825	1 250	5 215	5 175	10 390
			5 185	5 190	10 375
Mean = 48.261		Mean = 12.542	Mean = 51.906		

TAIL-WATER TITRATIONS.
Temp. 17.6° cent. Flask No. 11.

5 150	5 060	5 227	4 920	4 851	4 777
5 028		4 932		4 860	
5 185	5 100	5 145	5 015	5 093	5 005
	5 150		5 125		5 020

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper.....	50 794	48 520	49.783
Lower.....	51 450	50 516	50.931
Averages.	51.122	49.518	50.357 = Arith. mean.

Weighted mean = 50.664

*10 c.c. salt solution from each of Samples *B, D, F, I, K*, 50 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

TEST 211, OCTOBER 12TH, 1914.

TITRATION, OCTOBER 13TH.
Nitrate (1.56 grammes), October 12th.

Dilute salt solution. Temp. 18.25° cent Flasks Nos. 1 456-1 476.		Normal head-water. Temp. 13.8° cent. Flask No. 3.	Special dilutions.* Temp. 13.8° cent. Flasks Nos. 1 456-1 476.		
4 855	4 870	1 255	5 305	5 205	10 510
4 860	4 860	1 255	5 245	5 210	10 455
4 858	4 862	1 253	5 190	5 231	10 421
4 877	4 870	1 255	5 234	5 218	10 452
			5 248	5 235	10 483
Mean = 48.640		Mean = 12.545	Mean = 52.321		

TAIL-WATER TITRATIONS.
Temp. 16° cent. Flask No. 3.

5 160	5 140	5 165	4 915	4 848	4 787
5 111		4 974		4 850	
5 180	5 038	5 070	5 080	5 109	5 060
	5 070		5 111		5 100

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper.....	51 100	48 500	49.944
Lower.....	50 895	50 920	50.909
Averages	50.997	49.710	50.427 = Arith. mean.

Weighted mean = 50.636

*10 c.c. salt solution from each of Samples *B, D, F, I, K*, 50 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

Test 212, October 13th, 1914.

Titration, October 14th.
Nitrate (1.56 grammes), October 13th.

Dilute salt solution. Temp. 15.5° cent. Flasks Nos. 1 456–1 476.		Normal head-water. Temp. 11° cent. Flask No. 3.	Special dilutions.* Temp. 11° cent. Flasks Nos. 1 456–1 476.		
4 870	4 875	1 255	5 254	5 198	10 452
4 875	4 875	1 255	5 259	5 240	10 499
4 882	4 882		5 234	5 235	10 469
4 878	4 884		5 245	5 230	10 475
			5 245	5 215	10 460
Mean = 48.776		Mean = 12.550	Mean = 52.355		

* 10 c.c. salt solution from each of Samples *B*, *D*, *F*, *I*, *K*, 50 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

Tail-Water Titrations.
Temp. 13° cent. Flask No. 3.

5 477	5 476	5 402	5 205	5 165	5 100
5 490		5 230		5 109	
5 052	5 025	5 177	5 177	5 245	5 217
	5 050		5 240		5 235

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper.....	54 150	51 447	52.949
Lower.....	50 760	52 228	51.576
Averages.	52.455	51.838	52.262 = Arith. mean.
Weighted mean = 52.274			

It may be repeated that the tests are entered in consecutive order in the tables exactly as they were made in the field, the missing numbers, or letters, relating to other than regular turbine tests. Such, for example, are Tests *M*, which consisted of a series relating to the distribution of salt in head- and tail-races.

The concentrations of salt solution, normal head-water, special dilution, and tail-water are, of course, the mean values computed from the titration tables, Sections 103, 104, 105, and 106, or from similar tables on file as explained in Section 102.

In the case of the salt-solution, normal head-water, or special dilution samples, the simple arithmetical mean has been used, sometimes cutting out titrations which were questionable. The particular titrations thus cut out are enclosed in parentheses in the tables of titrations.

In the case of the tail-water, the weighted mean for the average titrations of the four quarters of the draft-tube has been used, the weights being determined from Figs. 37, 38, and 39. These weights are merely the four relative mean velocities for the four quarters of the draft-tube, as determined by the Ott current meters, uncorrected for their deviations from the still-water ratings.

Where the normal head-water has been titrated directly, a correction for the error of such a titration, as explained in Section 43 *et seq.*, has

TABLE 53.—Data for Computation of Turbine Discharge. Compiled from Records of Tests Made on Unit No. 11, Nov. 9th to Nov. 21st, 1914. See Table 52 for First Series of Tests on

TA FOR COMPUTATION OF TURBINE DISCHARGE. COMPILED FROM RECORDS OF TESTS MADE ON UNIT No. 11, NOV. 9TH TO NOV. 21ST, 1916

	Formula.	Symbol.	Units.	111	112	113	114	115	
			°C	5.7	5.0	4.5	4.5	4.6	
			°C	8.3	6.9	4.0	6.0	7.6	
			°C	8.8	8.9	6.9	4.0	9.4	
			°C	23.0	23.9	20.0	21.0	19.0	
			°C	6.6	7.3	9.6	7.8	8.6	
			°C	9.3	8.0	9.0	9.5	9.5	
			°C	8.4	8.0	8.3	8.5	7.5	
			°C	7.0	6.0	8.0	6.0	6.0	
	(Line 7 + Line 8) ÷ 2		°C	7.7	6.9	8.0	6.0	7.3	
			No.	9 171, 9 936	9 171, 9 936	9 171, 9 936	9 171, 9 936	9 171, 9 936	9 17
			c.c.	9.965	9.965	9.962	9.964	9.969	
			No.	1 407, 1 457	1 407, 1 457	1 407, 1 457	1 407, 1 457	1 407, 1 457	1 40
			c.c.	1 031.35	1 031.27	1 031.3	1 031.32	1 031.27	1
			c.c.	10.004	10.005	10.008	10.007	10.005	
			c.c.	59.944	61.975	62.005	62.052	54.944	
	Line 15 ÷ Line 14		c.c.	5 908.0	6 154.4	6 202.7	6 241.9	5 491.7	5
	Line 16 ÷ Line 17			100.46	100.65	100.51	100.49	100.44	
			c.c.	608 079	615 594	620 708	622 226	551 596	559
				201.04	209.30	214.68	214.63	270.79	
	$p = \frac{c}{d}$ (from curve)			25.50	25.76	25.15	26.15	23.40	
				1.1952	1.2013	1.2043	1.2047	1.1816	
Line 9				1.1952	1.2005	1.2050	1.2043	1.1824	
Line 9				1.1952	1.2000	1.2033	1.2049	1.1826	
				1.1952	1.2000	1.2034	1.2054	1.1826	
	Line 18 × Line 21 ÷ Line 22		c.c.	608 227	615 740	620 840	622 316	552 246	559
ature of Line 9	Line 18 × Line 24 ÷ Line 22	φ	c.c.	608 695	616 147	620 947	622 810	551 521	560
			div.	17	20	14	16	16	
			div.	26	34	43	31	53	
			sec.	1 926.3	1 934.1	1 936.4	1 935.7	1 936.3	1
	$\frac{0.498 \times (\text{Line } 28 - \text{Line } 27)}{\text{Line } 29}$		cu. ft.	0.97404	0.94209	0.94705	0.94254	0.94283	
erature of Line 9	Line 30 × Line 31 ÷ Line 34	φ	cu. ft.	0.97490	0.94289	0.94738	0.94281	0.94284	
ature of Line 9	Line 28 × Line 29 ÷ Line 34		c.c.	0.965	0.966	0.964	0.966	0.966	
				0.971	0.965	0.964	0.974	0.978	
				1 407, 1 457	1 407, 1 457	1 407, 1 457	1 407, 1 457	1 407, 1 457	1 407, 1
ities of salt solution				Δ (m = 5)	Δ (m = 5)	Δ (m = 5)	Δ (m = 5)	Δ (m = 5)	Δ (
			c.c.	3 028.91	3 028.94	3 028.15	3 028.0	3 028.1	3
				0.0000	0.0000	0.0000	0.0000	0.0000	
e 9				0.0000	0.0000	0.0000	0.0000	0.0000	
, temperature of Line 9	Line 35 × (Line 37 + Line 36)		c.c.	3 028.9	3 028.9	3 028.2	3 028.0	3 028.1	3
d and tail-water, temperature of Line 9	Line 39 + m × Line 38	v_0		60.338	60.369	60.321	60.326	60.321	
		h_0		0.973	0.971	0.970	0.970	0.975	
f head and tail-water, temperature of Line 9	Line 40 + Line 41	H_0		61.305	61.340	61.291	61.196	61.300	
		c.c.		9.965	9.965	9.965	9.964	9.965	
				1.0035	1.0035	1.0035	1.0035	1.0035	
				1.0064	1.0035	1.0035	1.0035	1.0035	
ure of Line 9	Line 43 × Line 44 ÷ Line 45		c.c.	9.994	9.998	9.994	9.994	9.994	
Line 9			c.c.	1 000.98	1 000.98	1 001.04	1 000.99	1 001.08	1
tail-water, temperature of Line 9	Line 47 × Line 37 ÷ Line 36		c.c.	1 001.0	1 001.1	1 001.04	1 001.1	1 001.0	1
il-water, temperature of Line 9	Line 48 ÷ Line 46	H_0		100.16	100.18	100.16	100.17	100.16	
e of Line 9	Line 49 × Line 42	E'		6 130.89	6 135.07	6 134.90	6 129.70	6 130.39	6
		h'		0.971	0.970	0.969	0.969	0.975	
	$E' - h'$	v'_1		6 129.93	6 134.05	6 132.93	6 128.73	6 129.43	6
		h		0.971	0.970	0.969	0.969	0.975	
				1 407, 1 457	1 407, 1 457	1 407, 1 457	1 407, 1 457	1 407, 1 457	1 40
		c.c.		500.49	500.49	500.52	500.49	500.51	
		c.c.		62.003	64.220	65.511	65.076	59.268	
	Line 56 ÷ Line 55	c.c.		125.095	128.515	130.597	130.594	117.796	
				0.9999	0.9999	0.9998	0.9998	0.9998	
				0.9998	0.9998	0.9998	0.9998	0.9998	
e of Line 9	Line 57 × Line 58 ÷ Line 54	v'_2	c.c.	125.070	128.503	130.577	130.574	117.796	
				5, 25, 33	5, 33	5	5	33	
		c.c.		500.51	500.50	500.52	500.52	500.51	
		c.c.		62.007	64.240	65.530	65.079	57.267	
	Line 62 ÷ Line 52	c.c.		125.045	128.540	130.572	130.579	114.594	
				0.9999	0.9998	0.9998	0.9998	0.9998	
				0.9998	0.9998	0.9998	0.9998	0.9998	
of Line 9	Line 64 × Line 66 ÷ Line 60	v_2	c.c.	125.046	128.540	130.572	130.579	114.594	
	Line 67 − Line 60	$v_2 - v'_2$		+ 10.974	+ 0.040	+ 0.005	— 1.845	— 3.202	
	Line 61 × Line 67	$h'v_2$		121.504	128.945	130.594	128.871	111.569	
	Line 68 − Line 69	$v - h'v_2$		001 703	615 021	620 620	622 608	551 419	559
				+ 62 902.6	+ 8 035.91	+ 1 748.45	— 2 470.89	— 15 195.8	— 42
	Line 80 × Line 69				+ 0.00836	+ 0.00278	— 0.01506	— 0.02552	
	Line 71 ÷ Line 70	$1 + \frac{f}{r}$		+ 0.10468	1.0084	1.0028	+ 0.98494	+ 0.97977	
	Line 72 + 1			1.1047					
	Line 38 ÷ Line 72	Q		5 542.40	6 113.26	6 115.80	6 332.44	6 305.13	6
	Line 81 × Line 74	E		1 524.70	1 497.54	1 515.79	1 510.67	1 535.95	1
	Same as Line 53			0.971	0.970	0.969	0.969	0.975	
	Line 31 × Line 76	$h_0 q$		0.967	0.936	0.935	0.935	0.935	
	Line 75 + Line 77	Q_0		1 525.0	1 497.5	1 516.2	1 511.1	1 536.2	1

$= h v_1$
$= V_1$

Value of v_1 is not given and was computed from Test 130 A as follows: $v_1 = \frac{E' v'_1 - v}{r}$ or $v_1 = \frac{6\,938.2 \times 114.478 - 598\,937}{6\,938} = 25.35$. Then $E' = \frac{594\,904 - 0.97 \times 25.33}{118.699 - 25.35} =$

TEST.

116	117	118	119	120 *A*	121 *A*	120 *B*	120 *B*	121 *B*	122 *B*
4.0 12.5 6.0 19.0 4.1 6.5 7.0 7.0 6.0 6.4 9 171, 9 936	4.0 9.0 7.0 19.0 7.7 8.0 6.0 8.0 6.0 7.0 9 171, 9 936	4.2 7.0 7.2 19.5 4.6 6.6 8.0 8.0 6.0 7.0 9 171, 9 936	2.5 5.0 7.2 19.5 4.6 6.6 8.0 8.0 6.0 7.0 9 171, 9 936	2.5 4.5 4.6 20.0 6.0 6.1 8.3 8.0 6.0 8.2 9 171, 9 936	2.5 5.0 4.6 21.5 7.5 6.0 8.3 8.0 6.0 8.2 9 171, 9 936	2.5 5.0 4.0 19.5 7.0 6.0 6.6 8.0 6.0 6.9 9 171, 9 936	2.5 20.0 4.0 20.5 6.0 13.6 6.0 8.0 6.0 8.2 9 171, 9 936	2.5 20.0 4.5 20.5 12.75 12.0 6.0 4.5 6.0 8.2 9 171, 9 936	2.0 12.5 6.0 14.0 8.0 12.5 4.5 4.0 6.0 6.3 9 171, 9 936
9.972 1 407, 1 407 1 091.7 10.005 58.774 5 905.5 100.49 520 197 2941.60 24.75	9.966 1 407, 1 476 1 031.55 10.027 58.593 6 211.0 100.51 630 045 3128.52 26.50	9.965 1 407, 1 407 1 031.57 10.035 58.141 6 211.0 100.46 624 281 3128.04 26.95	9.960 1 407, 1 407 1 031.59 10.036 58.141 6 150.1 100.59 615 279 3099.14 25.73	9.963 1 407, 1 407 1 031.3 10.035 58.489 5 999.4 100.43 596 749 2998.97 24.90	9.964 1 407, 1 407 1 031.34 10.035 59.025 5 905.7 100.50 599 888 3000.65 25.10	9.964 1 407, 1 407 1 028.52 10.004 58.163 b 214.3 100.49 594 094 307.01 23.70	9.970 1 407, 1 407 1 031.31 10.007 58.908 5 894.7 100.54 591 474 295.74 24.92	9.970 1 407, 1 407 1 031.31 10.007 58.908 5 906.1 100.54 601 247 300.63 25.50	9.978 1 407, 1 407 1 031.10 10.000 58.180 5 915.0 100.55 595 714 298.86 25.75
1.1025 1.1024 1.1023 1.1025 591 220 580 739 26 71 1 377.7' 0.23723	1.3028 1.3027 1.3027 1.3026 640 365 629 618 27 47 1 195.1 0.24055	1.3022 1.3021 1.3021 1.3020 624 643 624 260 26 47 1 191.6 0.24156	1.3022 1.3023 1.3023 1.3022 615 926 617 714 26 59 1 228.7 0.24116	1.1920 1.1920 1.1920 1.1916 597 219 595 597 21 63 1 215.4 0.24420	1.1922 1.1922 1.1922 1.1907 599 719 599 412 23 72 1 194.0 0.24103	1.1771 1.1771 1.1771 1.1768 584 094 588 597 27 68 1 197.2 0.21922	1.1954 1.1772 1.1547 1.1943 595 552 594 994 21 68 1 218.6 0.24450	1.1962 1.1960 1.1974 1.1974 602 201 604 975 23 72 1 194.0 0.24103	1.1778 1.1794 1.1778 1.1765 596 199 595 743 27 68 1 197.2 0.21922
0.24741 9.965 9.966 2, 23, 1 407, 1 427 A (m = 5) 3 038.2 0.9999 0.9999 3 038.6 60.342	0.24114 9.962 9.957 2, 23, 1 407, 1 427 C (m = 5) 3 038.04 0.9995 0.9999 3 031.8 67.035	0.24170 9.963 9.963 2, 23, 1 407, 1 427 C (m = 5) 3 038.0 0.9999 0.9999 3 039.0 66.968	0.24125 9.962 9.978 23, 1 407, 1 427 A (m = 5) 3 038.78 0.9999 0.9999 3 038.8 60.219	0.24449 9.954 9.969 23, 1 407, 1 427 C (m = 5) 3 031.5 0.9999 0.9999 3 031.0 66.595	0.24120 9.954 9.964 23, 1 407, 1 427 B (m = 4) 3 038.22 0.9999 0.9999 3 038.2 62.764	0.21940 9.954 9.978 23, 23, 1 407, 1 427 D (m = 5) 3 038.15 0.9999 0.9999 3 038.2 ....

been made. Where the normal head-water titration is computed from the salt-solution and special dilution titrations, no correction is necessary, except where there is a large relative difference, say 15% or even 20%, between the titrations of the salt-solution and special-dilution samples.

TEST 105, SEPTEMBER 12TH, 1914.

TITRATION, SEPTEMBER 13TH.

Nitrate (1.56 grammes), September 8th.

Dilute salt solution. Temp. 18° cent. Flasks Nos. 1 456-1 474.		Normal head-water. Temp. 18.25° cent. Flasks Nos. 3–11.	Special dilutions.* Temp. 18.25° cent. Flasks Nos. 1 456-1 474.		
5 310	5 350	1 310	4 775	4 820	9 595
5 350	5 355	1 305	4 835	4 805	9 640
5 350	5 360	1 320	4 810	4 790	9 600
5 355	5 360	1 315			
5 363	5 352	1 305			
5 365	5 347	1 320			
Mean = 53.514		Mean = 13.125	Mean = 48.058		

TAIL-WATER TITRATIONS.

Temp. 18.25° cent. Flask No. 11.

5 055	5 050	5 045	5 160	5 210	5 260
5 070		5 150		5 235	
6 230	6 208	6 718	6 700	6 770	6 770
	6 470		6 910		6 918

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper.....	50 740	52 162	51.372
Lower.....	64 052	68 136	66.321
Averages.	57.396	60.149	58.847 = Arith. mean.
Weighted mean = 57.920			

* Four 10-c.c. pipettes of salt solution from Samples ———, 40 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

TEST 106, SEPTEMBER 15TH, 1914.

TITRATION, SEPTEMBER 16TH.

Nitrate (1.56 grammes), September 8th.

Dilute salt solution. Temp 22° cent. Flasks Nos. 1 456-1 474.		Normal head-water. Temp. 18° cent. Flasks Nos. 3–11.	Special dilutions.* Temp. 21° cent. Flasks Nos. 1 456-1 474.		
4 340	4 335	1 305	4 150	4 150	8 300
4 335	4 340	1 309	4 120	4 165	8 285
4 335	4 335	1 305	4 190	4 095	8 285
4 335	4 335	1 302	4 147	4 135	8 282
4 335	4 335	1 295			
4 330	4 330	1 280			
Mean = 43.350		Mean = 12.993	Mean = 41.440		

TAIL-WATER TITRATIONS.

Temp. 21° cent. Flask No. 3.

4 650	4 730	4 725	4 710	4 615	4 720
4 690		4 750		4 775	
4 435	4 855	4 710	4 880	4 925	4 945
	4 560		4 905		4 950

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East	Averages.
Upper.....	4 709	4 705	47.072
Lower.....	4 640	4 921	47.961
Averages.	46.745	48.130	47.517 = Arith. mean.
Weighted mean = 47.087			

* Four 10-c.c. pipettes of salt solution from Samples ———, 40 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

TEST 107, SEPTEMBER 16TH, 1914.

TITRATION, SEPTEMBER 17TH.

Nitrate (1.56 grammes), September 8th.

Dilute salt solution. Temp. 19.5° cent. Flasks Nos. 1 456–1 474.		Normal head-water. Temp. 21° cent. Flasks Nos. 3–11.	Special dilutions.* Temp. 21° cent. Flasks Nos. 1 456–1 474.		
2 980	2 975	1 295	3 240	3 240	6 480
2 975	2 975	1 290	3 270	3 220	6 490
2 985	2 965	(1 350)	3 242	3 235	6 477
2 975	2 978	1 320	3 240	3 225	6 465
		(1 377)			
		1 310			
Mean = † 29.755		Mean = 13.038	Mean = 32.390		

TAIL-WATER TITRATIONS.

Temp. 20.5° cent. Flask No. 3.

4 055		4 015	4 070	3 975	4 010
	4 085				
4 090		4 075		4 025	
	4 511		4 757		4 795
4 345		4 660		4 800	
	4 508		4 837		4 855

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper.....	40 640	40 200	40.444
Lower.....	45 060	48 088	46.742
Averages.	42.850	44.144	43.593 = Arith. mean.

Weighted mean = 43.209

* Four 10-c.c. pipettes of salt solution from Samples———, 40 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

† 0.03 c.c. of $AgNO_3$ deducted from salt solution titration because that much was used up by the $K_2Cr_2O_7$, as the concentration of salt solution was weak for the ratio of dilution that was used in the laboratory on January 3d, 1915.

TEST 108, SEPTEMBER 17TH, 1914.

TITRATION, SEPTEMBER 19TH.

Nitrate (1.56 grammes), September 8th.

Dilute salt solution. Temp. 23° cent. Flasks Nos. 1 456–1 474.		Normal head-water. Temp. 19° cent. Flask No. 11.		Special dilutions.* Temp. 22.5° cent. Flasks Nos. 1 456–1 474.		
5 630	5 630	1 305	1 295	4 160	4 047	8 207
5 637	5 635	1 300	1 290	4 143	4 075	8 218
5 630	5 630	1 295	1 290	4 100	4 085	8 185
5 625	5 625			4 065	4 050	8 115
Mean = 56.303 †		Mean = 12.958		Mean = 40.906		

TAIL-WATER TITRATIONS.

Temp. 19° cent. Flask No. 11.

4 951		4 910	4 813	4 525	4 516
	4 905				
4 875		4 854		4 730	
	5 130		4 920		4 880
4 835		4 900		4 900	
	4 850		4 905		4 925

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper.....	48 990	46 460	47.865
Lower.....	49 288	49 060	49.161
Averages.	49.139	47.760	48.513 = Arith. mean.

Weighted mean = 48.778

* 40 c.c. salt solution from Sample *I*, diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

† Four 10-c.c. pipettes of salt solution diluted to 3 000 c.c. distilled water.

TEST 109, SEPTEMBER 18TH, 1914.

TITRATION. SEPTEMBER 19TH.

Nitrate (1.56 grammes), September 8th.

Dilute salt solution. Temp. 20° cent. Flasks Nos. 1 456-1 474.		Normal head-water, Temp. 20° cent. Flask No. 11.		Special dilutions.* Temp. 20° cent. Flasks Nos. 1 456-1 474.		
6 115	6 125	1 290	1 320	4 375	4 305	8 680
6 127	6 130	1 300	1 310	4 375	4 362	8 737
6 130	6 140	1 320	1 320	4 300	4 360	8 660
6 130	6 130			4 315	4 390	8 705
Mean = 61.284†		Mean = 13.100		Mean = 43.478		

*40 c.c. salt solution from Sample *I*, diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

†Four 10-c.c. pipettes of salt solution diluted to 3 000 c.c. distilled water.

TAIL-WATER TITRATIONS.

Temp. 22.5° cent. Flask No. 11.

5 170		5 130		5 170	4 790		4 625
5 130	5 150	5 130			4 870		
5 045	5 140	5 058		5 057	5 042		5 048
	5 045			5 060			5 070

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper	51 425	48 638	50.183
Lower	50 720	50 554	50.628
Averages.	51.072	49.596	50.406 = Arith. mean.

Weighted mean = 50.641

The pipette and flask calibrations are given in Figs. 4 and 6, respectively.

108.—Tests on Which Special Dilutions Were Used.—No special dilutions were used in the tests on Unit No. 13, except in the second treatments of Tests 23 and 26. In all the tests on Unit No. 12, and in the first series on Unit No. 11, special dilutions and normal head-water samples were both used so as to show that the corrections computed in Section 43 give accurate results when applied to normal head-water titrations. A comparison of the simultaneous results by the two methods is given in Section 86.

No normal head-water samples were titrated in the second series of tests on Unit No. 11.

109.—Final Computations.—The final computations, individually unadjusted except as to the calibrations of instruments and corrections indicated in the various tables, appear in Tables 54, 55, 56, 57, 58, and 59.

Table 54, Unit No. 13, First Series.—This is for the first, or lettered, series of tests on Unit No. 13. These tests are quite accurate as to discharge measurements, but cannot be regarded as anything but

preliminary tests for the purpose of adjusting the apparatus and training the corps of observers. The power was measured on the switchboard instruments only.

TEST 111, NOVEMBER 9TH, 1914.

TITRATION, NOVEMBER 10TH.

Nitrate (1.45 grammes), November 6th.

Dilute salt solution. Temp. 6.2° cent. Flasks Nos. 1 407-1 457.		Special dilutions.* Temp. 5.8° cent. Flasks Nos. 1 407-1 457.		
6 000	6 000	6 266	6 231	12 497
5 990		6 340	6 362	12 702
6 005	6 005	6 254	6 289	12 543
5 988	5 980			
6 000	5 982			
Mean = 59.944		Mean = 62.908		

TAIL-WATER TITRATIONS.

Temp. 8.3° cent. Flasks Nos. 3-22-28.

6 490	6 424	6 445	6 157	6 155	6 222
6 219		6 115		6 222	
7 284	7 443	7 288	7 208	7 258	7 200
	7 220		7 316		7 287

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper.....	63 386	61 890	62.721
Lower.....	73 088	72 538	72.782
Averages.	68.237	67.214	67.752 = Arith. mean.

Weighted mean = 68.007

* Five 10-c.c. pipettes of salt solution from Samples *A* and *E*, 50 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

TEST 112, NOVEMBER 10TH, 1914.

TITRATION, NOVEMBER 11TH.

Nitrate (1.45 grammes), November 6th.

Dilute salt solution. Temp. 6.9° cent. Flasks Nos. 1 407-1 457.		Special dilutions.* Temp. 8° cent. Flasks Nos. 1 407-1 457.		
6 141	6 152	6 480	6 437	12 917
6 186	6 187	6 498	6 478	12 976
6 150	6 163	6 455	6 564	13 019
6 160	6 171	6 464	6 480	12 944
6 128	6 137			
Mean = 61.575		Mean = 64.820		

TAIL-WATER TITRATIONS.

Temp. 8° cent. Flasks Nos. 3-28.

5 914	5 866	5 830	5 990	6 087	7 100
5 862		5 786		6 115	
7 135	6 827	7 382	7 068	7 420	7 353
	6 328		7 450		7 510

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper	58 516	63 230	60.611
Lower	69 180	73 602	71.637
Averages.	63.848	68.416	66.124 = Arith. mean.

Weighted mean = 64.946

* Five 10-c.c. pipettes of salt solution from Samples ———, 50 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

Table 55, Unit No. 13, Second Series.—In these tests the power was measured by the high-precision, direct-current instruments very satisfactorily, but there was some question toward the last as to the tail-

TEST 113, NOVEMBER 11TH, 1914.

TITRATION, NOVEMBER 12TH.

Nitrate (1.45 grammes), November 6th.

Dilute salt solution. Temp. 4° cent. Flasks Nos. 1 407-1 457.		Special dilutions.* Temp. 8.5° cent. Flasks Nos. 1 407-1 457.		
6 270	6 280	6 522	6 512	13 034
6 264	6 252	6 492	6 512	13 004
6 280	6 290	6 540	6 530	13 070
6 264	6 270	6 539	6 522	13 061
6 260	6 265			
Mean = 62.695		Mean = 65.211		

TAIL-WATER TITRATIONS.

Temp. 9° cent. Flask No. 3.

5 897	5 868	5 915	5 912	6 133	6 120
5 828		5 828		5 983	
7 086	7 206	7 205	7 200	7 444	7 352
	6 917		7 428		7 423

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper.....	58 672	60 370	59.427
Lower.....	70 910	73 694	72.457
Averages.	64.791	67.032	65.942 = Arith. mean.

Weighted mean = 65.320

* Five 10-c.c. pipettes of salt solution from Samples —— and *E*, 50 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

TEST 114, NOVEMBER 12TH, 1914.

TITRATION, NOVEMBER 13TH.

Nitrate (1.45 grammes), November 12th.

Dilute salt solution. Temp. 6° cent. Flasks Nos. 1 407-1 457.		Special dilutions.* Temp. 4° cent. Flasks Nos. 1 407-1 457.		
6 267	6 277	6 550	6 535	13 085
6 272	6 271	6 538	6 546	13 084
6 260		6 581	6 527	13 108
6 260	6 270	6 576	6 528	13 104
6 255	6 265			
Mean = 62.663		Mean = 65.476		

TAIL-WATER TITRATIONS.

Temp. 8.5° cent. Flask No. 3.

5 960	5 912	5 957	5 910	6 027	6 020
5 908		5 814		5 990	
6 795	6 928	7 145	7 020	7 410	7 304
	6 745		7 400		7 396

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper.....	59 102	59 868	59.442
Lower.....	69 033	73 060	71.270
Averages.	64.068	66.464	65.356 = Arith. mean.

Weighted mean = 64.672

* Five 10-c.c. pipettes of salt solution from Samples *A* and *C*, 50 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

water samples, owing to the discovery of leaks in the tail-water sampling system, in the lower section, at the elevation of the river water above the tail-water proper. This led to the transfer of the twelve tail-water sampling pipes from the lower to the upper sampling section, and to an increase in the number of regular samples from twelve to eighteen.

TEST 115, NOVEMBER 13TH, 1914.

TITRATION, NOVEMBER 14TH.

Nitrate (1.45 grammes), November 12th.

Dilute salt solution. Temp. 7.6° cent. Flasks Nos. 1 407-1 457.		Special dilutions.* Temp. 6.4° cent. Flasks Nos. 1 407-1 457.		
5 485	5 494	5 908	5 880	11 788
5 514	5 500	5 860	5 881	11 741
5 475	5 510	5 935	5 905	11 840
5 495	5 477	5 921	5 876	11 797
5 480	5 514			
Mean = 54.944		Mean = 53.958		

TAIL-WATER TITRATIONS.

Temp. 8.5° cent. Flask No. 22.

5 424	5 400	5 429		
5 365		5 320		5 488
5 312	5 260	5 404		
5 956		6 137		6 462
5 949	6 185	6 500		
5 950		6 517		6 560

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper.	53 522	54 102	53.780
Lower.....	60 100	64 352	62.462
Averages.	56.811	59.227	58.121 = Arith. mean.

Weighted mean = 57.457

*Five 10-c.c. pipettes of salt solution from Samples *A* and *D*, 50 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c., with normal head-water This liter then divided into two nearly equal parts, 500 c. c. each, for evaporation to about 10 c.c. in a casserole for titration.

TEST 116, NOVEMBER 14TH, 1914.

TITRATION. NOVEMBER 16TH.

Nitrate (1.45 grammes), November 12th.

Dilute salt solution. Temp. 18.5° cent. Flasks Nos. 1 407-1 457.		Special dilutions.* Temp. 6° cent. Flasks Nos. 1 407-1 457.		
5 860	5 877	6 215	6 220	12 435
5 875	5 872	6 186	6 213	12 399
5 875	5 864	6 248	6 173	12 421
5 875	5 878	6 178	6 248	12 426
5 855	5 883			
Mean = 58.714		Mean = 62.101		

TAIL-WATER TITRATIONS.

Temp. 9.5° cent. Flask No. 3.

5 650†	5 684	5 534		
5 617		5 558		5 578
5 577	‡5 519		‡5 519	
6 009		6 151		6 437
5 800	6 187	6 464		
5 998		6 348		6 500

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper.....	56 094	55 472	55.818
Lower.....	59 985	63 800	62.104
Averages.	58.039	59.636	58.961 = Arith. mean.

Weighted mean = 58.512

*Five 10-c.c. pipettes of salt solution from Samples *A* and *D*, 50 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

†Interpolated value—spoiled titration.

‡Titrated together.

Table 56, Unit No. 13, Third Series.—This is the best series of tests on any of the three units. The power was accurately measured by the high-precision, direct-current instruments.

The tail-race samples were very satisfactory, having been taken from eighteen points in the upper sampling station.

TEST 117, NOVEMBER 16TH, 1914.

TITRATION, NOVEMBER 17TH.

Nitrate (1.45 grammes), November 12th.

Dilute salt solution. Temp. 9° cent. Flasks Nos. 1 407–1 476.		Special dilutions.* Temp. 9° cent. Flasks Nos. 1 407–1 457.		
6 365	6 353	6 090	6 043	12 133
6 367	6 370	6 072	6 060	12 132
6 377	6 392	6 010	6 075	12 085
6 358	6 353	6 032	6 122	12 154
6 345	6 345			
Mean = 63.625		Mean = 60.630		

TAIL-WATER TITRATIONS.

Temp. 8.2° cent. Flask No. 28.

5 900	5 828		5 680		
5 783	5 788	5 717	5 800	5 800	
	5 670				
	5 885		6 486		6 670
6 127	6 320	6 520	6 668	6 720	6 665

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper.....	57 928	57 492	57.734
Lower.....	62 130	66 418	64.512
Averages.	60.029	61.955	61.123 = Arith. mean.

Weighted mean = 60.665

*Three 10-c.c. pipettes of salt solution from Samples *A* and *D*, 39 c.c., diluted with 2 000 c c. normal head-water, and 10 c.c. of this diluted with 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

TEST 118, NOVEMBER 17TH, 1914.

TITRATION, NOVEMBER 18TH.

Nitrate (1.45 grammes), November 12th.

Dilute salt solution. Temp. 7° cent. Flasks Nos. 1 407–1 457.		Special dilutions.* Temp. 7° cent. Flasks Nos. 1 407–1 457.		
6 239	6 215	6 005	5 991	11 996
6 240	6 230	6 025	6 015	12 040
6 205	6 190	5 965	6 025	11 990
6 231	6 215	5 970	6 000	11 970
6 206	6 170			
Mean = 62.141		Mean = 59.995		

TAIL-WATER TITRATIONS.

Temp. 8° cent. Flasks Nos. 3–22.

6 375	6 335		6 140		
6 250	6 169	6 123	6 172	6 252	6 298
	5 765		5 734		6 375
5 819	5 965	5 959	6 140	6 255	6 277

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper.....	62 504	62 155	62.349
Lower.....	58 770	61 562	60.321
Averages.	60.637	61.858	61.335 = Arith. mean.

Weighted mean = 61.016

*Three 10-c.c. pipettes of salt solution from Samples *A* and *D*, 30 c.c., diluted with 2 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each for evaporation to about 10 c.c. in a casserole for titration.

It should be remarked that Test 39 has been interpolated. The head-water elevations were lost, and, unfortunately, there was an unusual variation in this respect during the test. This was a disappointment, as it would have been desirable to know what degree of consistency such a test would show, as compared with those made under more favorable conditions. As it is, very little weight should be at-

Test 119, November 18th, 1914.

Titration, November 19th.

Nitrate (1.45 grammes), November 12th.

Dilute salt solution. Temp. 2° cent. Flasks Nos. 1 407–1 457.		Special dilutions.* Temp. 4.2° cent. Flasks Nos. 1 407–1 457.		
6 130	6 147	6 420	6 468	12 888
6 185	6 180	6 390	6 455	12 845
6 131	6 145	6 478	6 430	12 908
6 168	6 150	6 435	6 490	12 925
6 157	6 145			
Mean = 61.538		Mean = 64.458		

Tail-Water Titrations.

Temp. 4.6° cent. Flasks Nos. 3–32.

6 649	6 598	6 650	6 385	6 195	6 165
6 475		6 395		6 170	
6 258	6 295	5 867	5 875	5 821	5 930
	6 126		5 845		5 805

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper	65 534	62 288	64.091
Lower ...	61 852	58 552	59.797
Averages	63.443	60.420	61.944 = Arith. mean.
Weighted mean = 62.727			

* Five 10-c.c. pipettes of salt solution from Samples *A* and *E*, 50 c.c., diluted with 3 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

Test 120-*A*, November 19th, 1914.

Titration, November 20th.

Nitrate (1.45 grammes), November 19th.

Dilute salt solution. Temp. 4.2° cent. Flasks Nos. 1 407–1 457.		Special dilutions.* Temp. 4° cent. Flasks Nos. 1 407–1 457.		
5 845	5 845	5 738	5 732	11 470
5 840	5 840	5 725	5 735	11 460
5 840	5 853	5 750	5 710	11 460
5 850	5 830	5 744	5 701	11 445
Mean = 58.429		Mean = 57.294		

Tail-Water Titrations.

Temp. 6.1° cent. Flasks Nos. 3–22.

5 968	5 895	5 980	5 780	5 849	6 004
5 834		5 705		5 585	
5 547	5 678	5 425	5 503	5 684	5 765
	5 660		5 510		5 685

Calculations of Averages for Quarters and Halves of Race.

	West.	East.	Averages.
Upper	58 764	58 045	58.444
Lower	55 775	56 294	56.063
Averages	57.269	57.170	57.254 = Arith. mean.
Weighted mean = 57.244			

* Three 10-c.c. pipettes of salt solution from Samples *A* and *D*, 30 c.c., diluted with 2 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

tached to this test. The method of interpolating was to determine a head-water elevation which would satisfy the facts and agree with the other tests in a satisfactory manner.

Table 57, Unit No. 12.—These tests are not as satisfactory as the Third Series on Unit No. 13, there being some irregularities in both the power measurements and tail-water samples. The irregularities in

TEST 120-*B*, NOVEMBER 19TH, 1914.

TITRATION, NOVEMBER 22D.

Nitrate (1.45 grammes), November 19th.

Dilute salt solution. Temp. 20° cent. Flasks Nos. 1 407–1 457.		Special dilutions.* Temp. 4° cent. Flasks Nos. 1 407–1 457.		
5 914	5 920	5 929	5 960	11 889
5 918	5 913	5 931	5 940	11 871
5 899	5 903	5 960	5 900	11 860
5 900	....			
5 875	5 847			
Mean = 58.988		Mean = 59.367		

TAIL-WATER TITRATIONS.

Temp. 13.6° cent. Flask No. 22

6 000	5 977	5 897		
5 905		5 825	6 045	
5 860	5 758	....		
5 696		5 555	5 731	
5 603	5 440	5 640		
5 660		5 518	5 603	

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper.....	59 000	59 223	59.084
Lower.....	55 998	56 094	56.051
Averages.	57.499	57.658	57.478 = Arith. mean.

Weighted mean = 57.553.

* Three 10-c.c. pipettes of salt solution from Samples *A* and *D*, 30 c.c., diluted with 2 000 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

TEST 121-*A*, NOVEMBER 20TH, 1914.

TITRATION, NOVEMBER 21ST.

Nitrate (1.45 grammes), November 19th.

Dilute salt solution. Temp. 5° cent. Flasks Nos. 1 407–1 457.		Special dilutions.* Temp. 4.6° cent. Flasks Nos. 1 407–1 457.		
5 966	5 975	6 100	6 140	12 240
5 972	5 978	....	(6 092)	....
5 965	5 948	6 120	6 080	12 200
5 980	5 976	6 115	6 075	12 190
5 965	5 960			
Mean = 59.685		Mean = 61.050		

TAIL-WATER TITRATIONS.

Temp. 7.2° cent. Flasks Nos. 3–22.

6 645	6 618	6 285		
6 450		6 278	6 394	
6 470	6 260	6 450		
5 955		5 271	5 344	
5 582	5 365	5 253		
5 550		5 320	5 215	

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper.....	64 886	63 518	64.278
Lower.....	56 130	52 806	54.283
Averages.	60.508	58.162	59.280 = Arith. mean.

Weighted mean = 59.856.

* Four 10-c.c. pipettes of salt solution from Samples *A* and *D*, 40 c.c., diluted with 2 500 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

the power measurements may be attributed largely to a change in the personnel of the power observers and manipulators, and to the greater haste necessary in order to complete the work within the time limit of 2 weeks for this unit. The discrepancies in the discharge measurements result largely from uncertainties as to the proper weights for the various tail-water concentrations, which become relatively more

TEST 121-*B*, NOVEMBER 20TH, 1914.

TITRATION. NOVEMBER 26TH.

Nitrate (1.45 grammes), November 23d.

Dilute salt solution. Temp. 2° cent. Flasks Nos. 1 407-1 457.		Special dilutions.* Temp. 4.5° cent. Flasks Nos. 1 407-1 457.		
6 030	6 013	6 093	6 147	12 240
5 972	5 952	6 122	6 138	12 260
6 026	6 015	6 165	6 191	12 356
5 958	5 956	6 065	6 211	12 276
6 022	6 019			
Mean = 59.963		Mean = 61.415		

TAIL-WATER TITRATIONS.

Temp. 12.9° cent. Flask No. 22.

6 700	6 622		6 319	
6 490		6 342		6 430
6 489	6 308		6 495	
	5 970		5 311	5 373
5 605		5 371	5 295	
	5 600		5 360	5 265

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper.....	65 218	63 965	64.661
Lower.....	56 365	52 208	54.611
Averages.	60.792	58.586	59.636 = Arith mean.

Weighted mean = 60.177

* Four 10-c.c. pipettes of salt solution from Samples *A* and *D*, 40 c.c., diluted with 2 500 c.c. normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

TEST 122, NOVEMBER 21ST, 1914.

TITRATION, NOVEMBER 22D.

Nitrate (1.45 grammes), November 19th.

Dilute salt solution. Temp. 2° cent. Flasks Nos. 1 407-1 457.		Special dilutions.* Temp. 2° cent. Flasks Nos. 1 407-1 457.		
5 344	5 320	5 145	5 140	10 285
5 323	5 310	5 155	5 149	10 304
5 323	5 305	5 215	5 275	10 390
5 305	5 305	5 194	5 200	10 394
5 318	5 310			
Mean = 53.163		Mean = 51.716		

TAIL-WATER TITRATIONS.

Temp. 6° cent. Flasks Nos. 22-28.

5 190		(8 953)	5 060	
	(8 377)		(8 130)	5 159
4 940		5 340	5 080	
	(8 082)		(7 070)	(7 902)
5 130		4 988	(6 801)	
	5 195		(6 540)	(6 527)

CALCULATIONS OF AVERAGES FOR QUARTERS AND HALVES OF RACE.

	West.	East.	Averages.
Upper.....	51 566	50 997	51.282
Lower.....	51 043	†51 202	51.122
Averages.	51.304	51.100	51.202 = Arith. mean.

Weighted mean = 51.257

* *Special Dilution D.*—Five 10-c.c. pipettes of salt solution from Samples *A*, *B*, *C*, *D*, and *E*, 50 c.c., diluted to 3 500 c.c. with normal head-water, and 10 c.c. of this diluted to 1 000 c.c. with normal head-water. This liter then divided into two nearly equal parts, 500 c.c. each, for evaporation to about 10 c.c. in a casserole for titration.

† Value interpolated.

important for Unit No. 12 owing to the fact that the hydraulic conditions of the tests were not so uniform nor so satisfactory, from the standpoint of turbine driving. The efficiency is markedly less for this unit than for Unit No. 13, and this may be attributed to the relatively higher degree of turbulence of the head-water. It can scarcely be doubted that turbulence in the head-water has an effect on the efficiency of a water-wheel, though this would probably be small, relatively. Tests to ascertain the truth or falsity of this proposition are highly desirable.

Table 58, Unit No. 11, First Series.—These tests were started on September 12th, 1914, but it was found necessary to shut down for repairs on September 17th, before there had been time to get the equipment adjusted. Test 105 is wholly untrustworthy, as the governor was not blocked and the gate varied over a large range. The power was absorbed by a large water-rheostat, but this was improperly placed in the tail-water, which varied in elevation from moment to moment, thus changing the load and causing the variations of gate mentioned. The load was also affected by variations of salt content in the tail-water. The rheostat was not used in any test after the first one, Test 105.

The writer is not an advocate of rheostat tests for efficiency trials of hydro-electric units in place. It is very true that a uniform load can be secured in this way, but it is far better to secure tests under actual operating conditions. That such tests can be accurately conducted on either direct or alternating circuits is amply proved by the whole series which form the subject of this paper. Moreover, it is demonstrated herein that considerable variations of load do not affect seriously the results of a test which is properly conducted, especially if such variations are restricted within certain limits. One such test will check another under similar conditions of equal average load with surprising exactness.

Table 59, Unit No. 11, Second and Last Series.—These tests are more satisfactory than those on Unit No. 12, but not so good as those on Unit No. 13. When corrected for speed variations, the results agree very consistently, except in the case of Test 122, which must be discarded entirely. In preparing for this test it was found that all the available sample bottles were full from other tests. Accordingly, some bottles which had been set aside with salt solution were rinsed out and

TABLE 54.—FINAL COMPUTATIONS. TESTS OF UNIT NO. 13. TESTS *A*-*W*. FIRST SERIES. JULY 22D–AUGUST 13TH, 1914.

Missing Letters: *M* Represents a Distribution Test. *A*, *D*, and *E* Represent Preliminary Salt Tests. *B*, *C*, *F*, *J*, *P*, *Q*, *R* Represent Current-Meter Tests.

Date.	Test.	Duration, in minutes.	Gate Opening, in Inches.		Speed, in revolutions per minute.	Head, *h*, in feet.	Power at machine terminals, in kilowatts.	Discharge, in cubic feet per second.	Performance on 30-Ft. Head.				Over-all efficiency on Unit No. 13, in percentage.	Mean temperature of water, in degrees, centigrade.	Weight of water, in pounds per cubic foot.	Remarks.
			On reg. cyl.	Actual.					Speed, in revolutions per minute.	Power at mach. ter., in kilowatts.	Power at mach. ter., in horse-power.	Discharge, in cubic feet per second.				
1914																Shower before test started. Rough calculations indicate that the salt solutions of Tests *U* and *T* were interchanged.
8-10	*U*	30-0	13 1/4	5.00	105.0	33.95	3 622	1 440	98.7	3 008	4 032	1 354	87.58	18.8	62.334	
8-12	*V*	30-30	13 1/4	5 00	102.0	34.17	3 666	1 553	95.6	3 015	4 042	1 455	81.71	18.4	62.338	
8-13	*W*	20-0	14	5.24	101.8	33.41	3 681	1 556	96.5	3 132	4 198	1 474	83.77	18.8	62.334	
8-8	*S*	30-0	14 1/4	5.48	108.7	35.08	4 055	1 640	100.5	3 207	4 299	1 517	83.35	18.3	62.340	In this series of tests: All samples are titrated separately. Dosing pipes not plugged in middle. The power was measured by the switch-board instruments. There was some question as to the tail-water samples during some of the tests, owing to the discovery of leaks in the sampling pipes. Dosed tail sampling pipes on lower section. Discharge obtained by method of unbalanced evaporations. This unit generates direct current.
7-24 P. M.	*L*	15-50	14.55	5.50	108.6	35.05	4 083	1 647	100.5	3 233	4 334	1 524	83.64	19.4	62.326	
7-22	*G*	24-45	"	"	108.0	35.10	4 078	1 662	99.8	3 222	4 319	1 536	82.70	19.1	62.330	
7-24 A. M.	*H*	16-0	"	"	108.0	35.24	4 112	1 662	99.7	3 230	4 330	1 534	83.02	19.3	62.327	
7-24 A. M.	*I*	16-0	14.6	5.52	107.8	35.30	4 115	1 666	99.4	3 224	4 322	1 536	82.76	19.4	62.326	
7-21 P. M.	*K*	18-0	14.55	5.50	108.1	34.97	4 069	1 683	100.1	3 233	4 334	1 558	81.82	19.4	62.326	
8-10	*T*	45-0	14 1/4	5.48	108.7	35.36	4 120	1 746	100.1	3 220	4 316	1 609	78.89	18.6	62.336	
7-30	*O*	39-0	16 1/8	6.26	108.0	34.98	4 333	1 768	100.0	3 442	4 614	1 637	82.90	18.6	62.336	
7-28	*N*	39-0	16 1/8	6.26	107.8	34.53	4 234	1 777	100.5	3 429	4 597	1 656	81.65	19.0	62.331	
														Ave..	62.332	

used, but the tail-water samples show conclusively that the bottles were not sufficiently cleansed. Some of the samples titrated nearly twice as high as they should. Certain samples, however, had the appearance of being about right in respect to their salt content, and the test has been worked up on this basis, with the result that the final figures are not far from the truth, but the facts rob the result of any intrinsic value. Test 118 seems to agree with Test 122, but an inspection will show that there was considerable difference (3 rev. per min.) in speed. The tests, therefore, are not directly comparable.

110.—Plottings of the Tests.—The final adjustment of the tests for the purpose of determining turbine efficiencies cannot now be made, as the efficiencies of the generators are not known for the various conditions of performance. The over-all efficiencies of the units, however, would appear to be well determined by the curves which may be drawn with reference to the final computations, Section 109.

Plate XLV, Unit No. 13, First Series.—These were the lettered tests in which the power was measured by the switch-board meters only. Although the discharge measurements were made with considerable accuracy, there is a strong probability that some carelessness in the laboratory resulted in the interchange of salt-solution samples for at least one pair, and possibly two pairs of tests. The most questionable pair in this respect is *T, U,* where an interchange of salt-solution samples would make better harmony as regards results. The fact which lends weight to this view is that at about this time the salt-solution samples were, on several occasions, titrated on the morning following the titration of the other samples for the same test—a procedure which is strongly to be condemned. Tests *T* and *U* are subject to this suspicion. They would be far more harmonious if the salt-solution titrations were interchanged in the calculations, as may be seen by an inspection of the data and results.

The power on these lettered tests may be rendered comparable with the later series by applying a correction to the switch-board instruments.

Plate XLVI, Unit No. 13, Second Series.—In this series, computed in Table 55, Section 109, the tail-race samples were taken principally from the lower sampling section. It has been mentioned that some leaks were discovered in the risers for these samples at points where

TABLE 55.—(*Continued.*)

Date.	Test	Duration, in minutes.	Gate Opening, in Inches.		Speed, in revolutions per minute.	Head, *h*, in feet.	Net power at machine terminals, in kilowatts.	Discharge, in cubic feet per second.	Performance on 30-Ft. Head.				Over-all efficiency on Unit No. 13, in percentage.	Mean temperature of water, in degrees, centigrade.	Weight of water, in pounds per cubic foot.	Remarks.
			On reg. cyl.	Actual.					Speed, in revolutions per minute.	Power at machine terminals, in kilowatts.	Power at machine terminals, in horse power.	Discharge, in cubic feet per second.				
1914.																
8-15	10	30	14	5.24	105.0	33.75	3 767	1 586 1 579	99.0	3 157	4 232	1 496 1 488	83.20 83.65	19.0	62.331	Showers at times. Apparently a good test chemically. All samples titrated separately.
8-21	20	18	14½	5.48	97.5	32.87	3 660	1 595 1 581	93.1	3 192	4 279	1 524 1 510	82.58 83.35	18.8	62.334	½ test chemically. E. and W. D. T. samples combined.
8-21	21	18	14½	5.48	101.8	32.80	3 681	1 589 1 582	97.4	3 219	4 315	1 520 1 513	83.50 83.88	18.8	62.334	½ test chemically. E. and W. D. T. samples combined, except Nos. 21 and 22, titrated separately.
8-20	18	18	15	5.72	117.1	33.61	3 914	1 636 1 609	110.6	3 301	4 425	1 546 1 520	84.18 85.62	18.6	62.336	Head-water muddy. Samples 21 and 22 titrated separately. Combined 23 with 24, 25 and 27, 26 and 28, 29 and 31, and 30 and 32. No. 22 bad sample.
8-18	11	30	15	5.72	106.1	34.11	4 018	1 661 1 652	99.5	3 314	4 442	1 558 1 549	83.86 84.34	18.8	62.334	Slight rain during test. Apparently good test chem. Samples titrated separately. No. 22 doubtful.
8-20	17	18	15	5.72	111.1	33.63	3 980	1 650 1 645	104.9	3 353	4 495	1 559 1 554	84.80 85.07	18.6	62.336	Head-water muddy. Samples 21 and 22 titrated separately. Combined 23 and 24, 25 and 27, 26 and 28, 29 and 31, 30 and 32. No. 22 bad sample.
8-18	12	30	15½	5.96	107.2	34.83	4 220	1 731 1 726	99.5	3 373	4 522	1 606 1 601	82.82 83.07	18.8	62.334	Apparently good test chem. Samples titrated separately. No. 22 doubtful.
														Ave.=	62.333	used on this series of tests.

General Notes.—This unit generates direct current. The flow of salt solution was not stopped between Tests 7 and 8, 13 and 14, 15 and 16, 17 and 18, 20 and 21, and each pair of tests was run in immediate succession. "East and West Dosed Tail Samples Mixed" means that each sample taken on the east side of draft-tube was mixed with its corresponding west sample taken at same elevation. Thus there were half as many samples to titrate as were actually taken out of the draft-tube.

In these tests the power was measured by the high-precision, direct-current instruments, and was very satisfactory, but there was some question as to the tail-water samples during some of the tests, owing to the discovery of leaks in the sampling pipes in the lower section at the elevation of river water above the tail-water proper. The upper figures in discharge and efficiency columns, are results obtained by taking titration of doubtful samples as correct. The lower figures are obtained by omitting the titration of doubtful samples, values being substituted, instead, which are based on the titration of adjacent samples.

Dosed tail-sampling pipes on lower section. Eight samples on upper section also in Test 19. Gate locked on all tests.

Discharge obtained by method of unbalanced evaporations.

TABLE 55.—FINAL COMPUTATIONS. TESTS OF UNIT NO. 13. TESTS 6-21, INCLUSIVE. SECOND SERIES.
AUGUST 13TH–AUGUST 21ST, 1914.
Missing Tests, 1-5, Inclusive, Represent Current-Meter Tests.

Date.	Test.	Duration, in minutes.	Gate Opening, in Inches.		Speed, in revolutions per minute.	Head, h, in feet.	Net power at machine terminals, in kilowatts.	Discharge, in cubic feet per second.	Performance on 30-Ft. Head.				Over-all efficiency on Unit No. 13, in percentage.	Mean temperature of water, in degrees, centigrade.	Weight of water, in pounds per cubic foot.	Remarks.
			On reg. cyl.	Actual.					Speed, in revolutions per minute.	Power at machine terminals, in kilowatts.	Power at machine terminals, in horse-power.	Discharge, in cubic feet per second.				
1914.																
								1 424				1 340	83.91			
8-19	13	18	12⅞	4.70	97.2	33.84	3 417	1 431	9.15	2 852	3 823	1 347	83.48	18.8	62.334	Head-water muddy. ⅓ test chem. E. and W. D. T. samples mixed. Salt soln. not turned off between Tests 13 and 14.
8-19	14	18	12⅞	4.70	102.0	33.92	3 420	1 438	96.0	2 845	3 814	1 353	82.91	18.8	62.334	Head-water muddy. ⅓ test chemically. E. and W. D. T. samples mixed.
8-15	9	28	12⅞	4.70	107.0	34.42	3 548	1 460	99.9	2 887	3 870	1 363	83.51	19.2	62.329	Apparently a good test chem. Samples titrated separately.
								1 522				1 417	84.50			
8-14	8	20	13½	5.00	113.0	34.60	3 762	1 514	105.2	3 037	4 071	1 410	84.92	19.0	62.331	Rain before test. Weighting imperfect. ⅓ test chemically. E. and W. D. T. samples mixed.
								1 517				1 432	83.55			
8-20	19	30	13½	5.00	106.0	33.69	3 612	1 507	100.0	3 035	4 068	1 422	84.14	18.8	62.334	Head-water very dirty. Apparently good test chemically. Samples titrated separately. 12 D. T. samples on lower sec. 8 on upper.
8-14	7	20	13½	5.00	100.0	34.72	3 851	1 556	93.0	3 093	4 146	1 446	84.33	19.0	62.331	Rain before test. ⅓ test chemically. E. and W. D. T. samples mixed.
8-19	16	18	14	5.24	112.0	32.81	3 559	1 526	107.1	3 111	4 170	1 459	84.06	18.9	62.333	Head-water muddy. ⅓ test chemically. E. and W. D. T. samples mixed.
								1 555				1 486	82.91			
8-19	15	18	14	5.24	108.0	32.88	3 585	1 548	103.1	3 125	4 189	1 478	83.36	18.9	62.333	⅓ test chemically. E. and W. D. T. samples mixed.
8-13	6	20	14	5.24	103.1	33.65	3 760	1 571	97.4	3 165	4 243	1 483	84.15	19.2	62.329	Rain before test. Apparently a good test chemically. All samples titrated separately.

contamination would be likely to occur. Hence, in computing Table 55, the titrations of the samples which appeared to be too low were at first corrected by cutting them out and inserting the averages of some of the adjacent samples in their places. The next diagram gives a plotting of the tests without any arbitrary corrections, and these check the third series even better than the plottings of the diagram now in question.

Tests 6, 9, 10, 11, 12, and 19 are the only ones of this series in which the tail-water samples were titrated in complete detail. In all the remaining tests the twelve regular tail-water samples were combined in pairs, by mixing equal parts of each one of every pair of samples, thus reducing the number of titrations for these samples from twelve to six. The small number of samples, however, is not the worst feature. The main trouble is that, with so few samples, it is not possible to correct for non-uniformity of distribution with any degree of certainty.

It is also to be observed that these particular tests were intended to cover a larger range of speed variations than was permissible under the terms of the contract. It was hoped by the writer that enough material could be accumulated in this way to diagram the wheels. Unfortunately, the representatives of the Allis-Chalmers Company did not agree unreservedly to such a plan, and it was necessary, therefore, to confine the work as closely as possible to the contract conditions of 100 rev. per min. On account of these speed variations, coupled with the uncertainties as to distribution of salt in the tail-race under such conditions, especially in the case of the partial tests just mentioned, it must not be expected that the points for the tests will plot in a smooth curve.

Test 19 of this series, and Tests 22 and 23 of the next, or third, series are worthy of particular notice, as the full quota of twelve samples was taken at the lower section while eight well-distributed samples were taken in the upper section, about 8 ft. farther up stream, and well away from any influence of unsalted tail-water. These were the last tests for which any samples were taken in the lower section. This was on account of the difficulty of maintaining the sampling system there, so as to prevent leaks in the pipes. Tests 20 and 21 were merely partial tests, as explained. After cutting out one sample on the lower section in Test 22 and three in Test 23, on account of breaks in the correspond-

TABLE 56.—Final Computations. Tests of Unit No. 13. Tests 22-46, Inclusive. Third Series. August 21st–September 2d, 1914.

Missing Tests, 24, 25, 27-31, Inclusive, and 34-38, Inclusive, Represent Current-Meter Tests.

Date.	Test.	Duration, in minutes.	Gate Opening, in Inches.		Speed, in revolutions per minute.	Head, h, in feet.	Power at machine terminals, in kilowatts.	Discharge, in cubic feet per second.	Performance on 30-Ft. Head.				Over-all efficiency on Unit No. 13, in percentage.	Mean temperature of water, in degrees, centigrade.	Weight of water, in pounds per cubic foot.	Remarks
			On reg. cyl.	Actual.					Speed, in revolutions per minute.	Power at mach. ter., in kilowatts.	Power at mach. ter., in horse-power.	Discharge, in cubic feet per second.				
1914																
9-2	46	25	13½	5.0	102	31.30	3 239	1 466	99.9	3 089	4 074	1 435	83.52	20.2	62.315	Dosed tail sampling pipes on upper section. Tests 26-46, both inclusive. Dosed tail sampling pipes on lower section. Tests 22-23. Eight samples also taken on upper section, Tests 22-23.
8-22	28	30	13¾	5.12	98.0	28.76	2 912	1 431	100.0	3 102	4 158	1 461	83.70	18.5	62.337	
8-27	33	30	13⅞	5.18	101.0	31.18	3 289	1 491	99.1	3 104	4 161	1 463	83.65	18.7	62.335	
9-1	44	25	13 15/16	5.20	101.0	30.77	3 262	1 497	99.8	3 140	4 209	1 479	83.71	19.6	62.323	This unit generates direct current. Gate locked on all tests. Discharge obtained by method of unbalanced evaporations. All samples titrated separately. Dosing pipes not plugged in middle.
8-31	41	30	14⅛	5.30	102.0	31.46	3 387	1 520	99.6	3 154	4 228	1 484	83.80	18.8	62.334	
8-26	26	30	14	5 24	100.0	29.60	3 102	1 482	100.7	3 165	4 243	1 492	83.68	18.4	62.338	
8-31	42	25	14¼	5 36	103.0	31.88	3 473	1 542	100.0	3 170	4 249	1 496	83.54	19.0	62.331	
8-21	22	30	14¼	5 36	99.7	29.72	3 148	1 496	100.0	3 192	4 279	1 503	83.74	19.0	62.331	
8-26	32	30	14⅛	5 48	101.0	31.41	3 442	1 552	98.7	3 213	4 307	1 517	83.50	18.7	62.335	
8-28	39	30	14¼	5.36	99.5	29.07	3 068	1 495	101.0	3 216	4 311	1 518	83.52	18.6	62.336	Head computed. Part of record lost.
9-2	45	25	14½	5.48	103.3	32.50	3 631	1 582	99.4	3 220	4 316	1 520	83.54	20.2	62.315	
9-1	43	25	14¾	5.60	99.5	29.76	3 219	1 536	100.0	3 258	4 367	1 542	83.30	19.4	62.326	
8-28	40	30	15	5.72	98.0	28.76	3 099	1 532	100.0	3 301	4 425	1 564	83.22	19.0	62.331	Pen on ammeter caught.

TABLE 57.—FINAL COMPUTATIONS. TESTS OF UNIT No. 12. TESTS 200-212, INCLUSIVE.
SEPTEMBER 28TH–OCTOBER 13TH, 1914.

Date.	Test.	Duration, in minutes.	Gate Opening, in Inches.		Speed, in revolutions per minute.	Head, h, in feet.	Net power at machine terminals, in kilowatts.	Discharge, in cubic feet per second.	Performance on 30-Ft. Head.				Over-all efficiency on Unit No. 12, in percentage.	Mean temperature of water, in degrees, centigrade.	Weight of water, in pounds per cubic foot.	Remarks.
			On reg. cyl.	Actual.					Speed, in revolutions per minute.	Power at machine terminals, in kilowatts.	Power at machine terminals, in horse-power.	Discharge, in cubic feet per second.				
1914.																
9-30	202	28	13 1/2	5.20	101.7	30.17	3 082	1 476	101.4	3 056	4 097	1 471	81.86	15.6	62.368	All samples except on Test 201 are titrated separately. Dosing pipes plugged in middle.
10-6	206	30	13 5/8	5.26	100.0	30.55	3 207	1 492	99.1	3 121	4 183	1 479	83.12	13.3	62.389	Dosed tail sampling pipes on upper section. These tests are not as satisfactory as the third series on Unit No 13, there being more irregularity in tail-water samples. The discharge was obtained by the methods of Unbalanced Evaporations and Special Dilutions, the average of the two being used.
10-7	207	34	13 5/8	5.26	102.0	30.56	3 179	1 487	101.1	3 092	4 145	1 473	82.71	13.27	62.389	
10-13	212	30	13 5/8	5.26	101.5	30.41	3 160	1 491	100.8	3 096	4 150	1 481	82.86	13.55	62.387	
9-29	201	30	13 3/4	5.32	102.0	31.05	3 336	1 515	100.3	3 168	4 246	1 490	83.76	16.1	62.363	This unit generates direct current. On Tests 200 to 204, inclusive, sample pipe No. 27 was broken, sample No. 27 actually came from top of upper draft-tube. Value for this sample interpolated. Pipe repaired after Test 204. Dosed tail samples averaged chemically in pairs, on Test 201, No. 43 with No. 51, No. 29 with No. 31, No. 55 with No. 41. No. 25 with No. 42, No. 28 with No. 30, No. 22 with No. 44. Gate locked on all tests. Rain during Tests 201 and 211.
10-2	204	30	13 3/4	5.32	100.3	30.92	3 287	1 505	98.8	3 141	4 210	1 482	83.49	15.2	62.372	
10-8	208	25	13 3/4	5.32	100.5	30.24	3 160	1 499	100.1	3 123	4 186	1 493	82.41	14.0	62.383	
10-12	211	25	13 3/4	5.32	103.0	31.85	3 389	1 545	100.0	3 098	4 153	1 499	81.43	14.0	62.388	
10-5	205	30	13 7/8	5.38	101.6	30.21	3 181	1 512	101.1	3 148	4 220	1 506	82.36	14.25	62.381	
10-9	209	25	13 7/8	5.38	100.2	31.12	3 330	1 561	98.4	3 152	4 225	1 532	81.05	14.1	62.382	
10-10	210	25	13 7/8	5.38	102.4	31.26	3 377	1 541	100.3	3 174	4 255	1 510	82.82	14.5	62.379	
10-1	203	25	14	5.44	99.5	30.62	3 275	1 519	98.5	3 176	4 257	1 504	83.19	15.3	62.371	
9-28	200	25	14 1/4	5.56	101.0	30.65	3 341	1 568	99.9	3 235	4 337	1 551	82.19	16.05	62.364	
														Ave.	62.378	(Used on this series of tests.)

TABLE 58.—FINAL COMPUTATIONS. TESTS OF UNIT NO. 11. TESTS 105-109, INCLUSIVE. FIRST SERIES.
SEPTEMBER 12TH–SEPTEMBER 18TH, 1914.
Tests 101-104, Inclusive, are Distribution Tests.

Date.	Test.	Duration, in minutes.	Gate Opening, in Inches.		Speed, in revolutions per minute.	Head, h, in feet.	Net power at machine terminals, in kilowatts.	Discharge, in cubic feet per second.	Performance on 30-Ft. Head.				Over-all efficiency on Unit No. 11, in percentage.	Mean temperature of water, in degrees, centigrade.	Weight of water, in pounds per cubic foot.	Remarks.
			On reg. cyl.	Actual.					Speed, in revolutions per minute.	Power at machine terminals, in kilowatts.	Power at machine terminals, in horse-power.	Discharge, in cubic feet per second.				
1914																
9-12	105	15	13.92	5.10	100.2	30.36	3 218	1 446	99.6	3 161	4 237	1 437	86.70	17.0	62.354	Unit on governor. Water rheostat connected on this test only.
9-18	109	20	14.0	5.14	102.7	31.23	3 293	1 487	100.7	3 101	4 157	1 458	83.84	18.0	62.343	
9-17	108	20	14¼	5.26	103.5	30.98	3 280	1 482	101.8	3 125	4 189	1 458	84.48	17.8	62.346	Dredge started during test. Head varied.
9-15	106	20	14½	5.38	103.4	31.99	3 561	1 539	100.1	3 233	4 334	1 490	85.53	17.2	62.352	Valves closed on Dosing Pipes Nos. 3, 4. 9, and 10, because Pipes 3 and 4 broke just before starting test.
9-16	107	20	14¾	5.50	103.0	30.78	3 388	1 519	101.7	3 260	4 370	1 500	85.66	17.6	62.348	
														Ave.=	62.349	

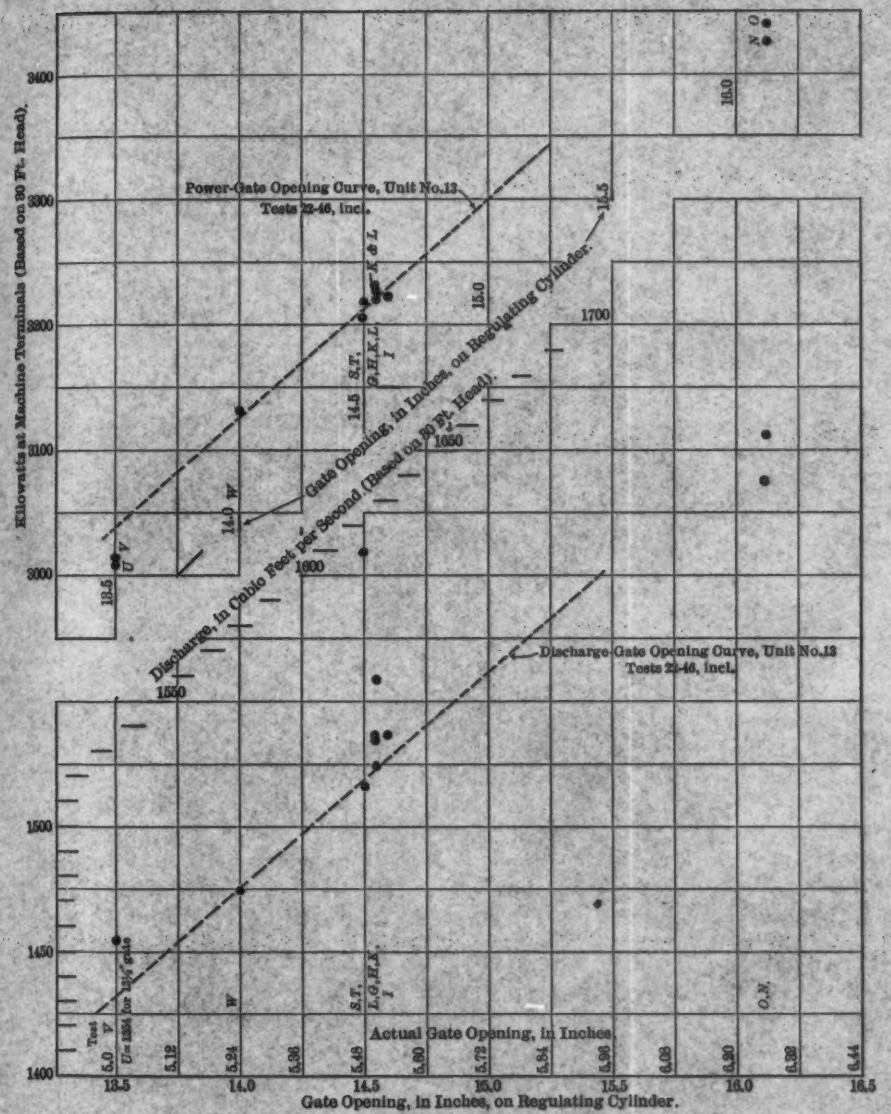
NOTES.—All samples titrated separately. Dosing pipes plugged in middle. No excitation power readings taken. Dosed tail sampling pipes on upper section. Discharge obtained by two methods: Unbalanced Evaporations and Special Dilutions. Average of the two results used. This unit generates alternating current. Gates locked on all tests except 105.

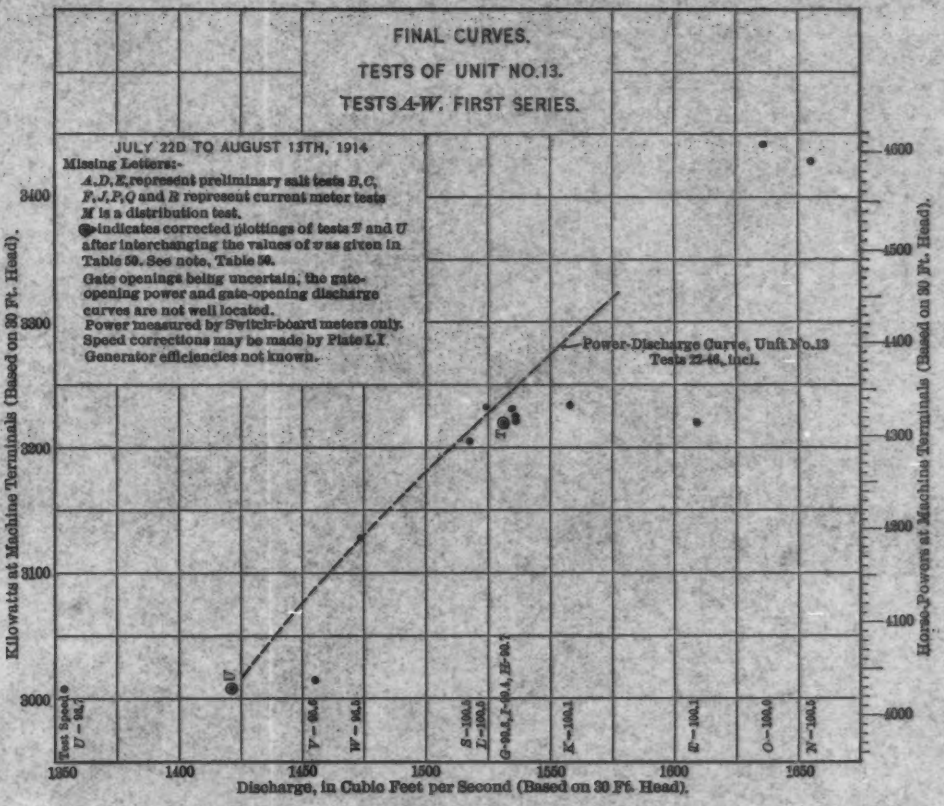
TABLE 59.—FINAL COMPUTATIONS. TESTS OF UNIT NO. 11. TESTS 111-122, INCLUSIVE. SECOND SERIES.

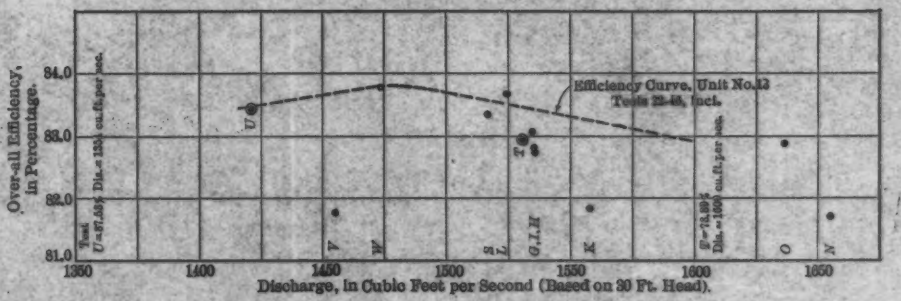
NOVEMBER 9TH–NOVEMBER 21ST, 1914.

No Test Run on No. 110.

Date.	Test.	Duration, in minutes.	Gate Opening, in Inches.		Speed, in revolutions per minute.	Head, h, in feet.	Net power at machine terminals, in kilowatts.	Discharge, in cubic feet per second.	Performance on 30 Ft. Head.				Over-all efficiency on Unit No. 11, in percentage.	Mean temperature of water, in degrees, centigrade.	Weight of water, in pounds per cubic foot.	Remarks.
			On reg. cyl.	Actual.					Speed, in revolutions per minute.	Power at machine terminals, in kilowatts.	Power at machine terminals, in horse-power.	Discharge, in cubic feet per second.				
1914.																
11-10	112	20	14 3/8	5.32	103.5	30.51	3 161	1 488	102.6	3 082	4 131	1 475	82.25	8.0	62.421	All samples titrated separately. Dosing pipes plugged in middle. Dosed tail sampling pipes on upper section.
11-11	113	20	14 3/8	5.32	103.0	31 84	3 413	1 513	100.0	3 121	4 184	1 469	83.65	8.0	62.421	Discharge obtained by special dilution method. This unit generates alternating current. Gate locked on all tests.
11-12	114	20	14 5/8	5.44	102.0	30 88	3 312	1 511	100.5	3 171	4 251	1 489	83.85	8.0	62.421	
11-9	111	20	14 5/8	5.44	106.9	31 36	.....	1 525	104.6	.....	.....	1 492	.... .	7.7	62.422	No power readings taken.
11-18	119	20	14 3/4	5.50	105.5	30 99	3 339	1 531	103.8	3 180	4 263	1 507	83.08	5.0	62.428	
11-13	115	20	14 7/8	5.56	103.5	31 31	3 419	1 536	101.3	3 206	4 298	1 504	83.93	7.8	62.422	
11-14	116	20	15	5.62	104.0	32 24	3 607	1 572	100.3	3 237	4 339	1 516	84.06	6.4	62.426	Rained hard during test.
11-20	121	20	15 1/8	5.68	103.5	31.28	3 483	1 580	101.4	3 271	4 385	1 547	83.25	4.8	62.428	Snowed during test.
11-17	118	20	15 1/4	5.74	103.7	33.56	3 889	1 609	98.0	3 287	4 406	1 521	85.08	6.0	62.427	
11-21	122	20	15 1/4	5.74	103.8	31.67	3 581	1 563	101.0	3 302	4 426	1 521	85.46	4.2	62.428	Tail-race samples spoiled.
11-16	117	20	15 1/8	5.86	105.5	32.90	3 899	1 639	100.7	3 334	4 469	1 565	83.87	7.0	62.424	
11-19	120	20	15 3/4	5.98	105.5	32.28	3 784	1 664	101.7	3 390	4 544	1 605	83.15	5.2	62.428	
														Ave.=	62.425	(Used on this series of tests.)


Power-Gate Opening Curve, Unit No.13
Tests 22-46, incl.
Discharge-Gate Opening Curve, Unit No.13
Tests 22-46, incl.
Gate Opening, in Inches, on Regulating Cylinder.
Discharge, in Cubic Feet per Second (Based on 30 Ft. Head).
Kilowatts at Machine Terminals (Based on 30 Ft. Head).
Actual Gate Opening, in Inches.
Gate Opening, in Inches, on Regulating Cylinder.


FINAL CURVES.
TESTS OF UNIT NO.13.
TESTS A-W. FIRST SERIES.
JULY 22D TO AUGUST 13TH, 1914
Missing Letters:-
A, D, E, represent preliminary salt tests B, C, F, J, P, Q and R represent current meter tests
M is a distribution test.
⊕ indicates corrected plottings of tests T and U after interchanging the values of v as given in Table 50. See note, Table 50.
Gate openings being uncertain, the gate-opening power and gate-opening discharge curves are not well located.
Power measured by Switch-board meters only.
Speed corrections may be made by Plate L.I.
Generator efficiencies not known.
Power-Discharge Curve, Unit No.13
Tests 22-46, incl.
Kilowatts at Machine Terminals (Based on 30 Ft. Head).
Horse-Powers at Machine Terminals (Based on 30 Ft. Head).
Discharge, in Cubic Feet per Second (Based on 30 Ft. Head).


Efficiency Curve, Unit No.13
Tests 22-46, incl.
Over-all Efficiency, in Percentage.
Discharge, in Cubic Feet per Second (Based on 30 Ft. Head).

ing pipes, Table 60 was compiled. It shows an interesting comparison of titrations of the two sets of samples in these tests.

TABLE 60.—Comparison of the Average Titrations from the Upper Sampling Section with the Average from the Lower Section.

(The averages are from eight samples on the upper section and twelve on the lower, except that one sample on the lower section for Test 22, and three for Test 23, have been suppressed on account of leaks in the corresponding pipes.)

Test 19.

	Upper Section.		Lower Section.	
	W.	E.	W.	E.
Upper half	58.41	58.07	58.22	58.30
Lower half	58.55	59.08	58.71	59.12
Means	58.48	58.58	58.46	58.71

Test 22.

Upper half	59.45	59.52	59.05	58.66
Lower half	57.65	57.90	58.20	58.10
Means	58.55	58.71	58.62	58.38

Test 23.

Upper half	60.89	60.72	60.26	60.12
Lower half	57.06	56.95	56.52	58.10
Means	58.98	58.84	58.39	59.11

Summary of Means.

Test.	Average upper section.	Average lower section.
19	58.53	58.58
22	58.63	58.50
23	58.91	58.75
Mean	58.69	58.61

From Table 60, therefore, it is apparent that there was no material difference between the concentrations from the upper and lower sta-

tions when the pipes on the lower station were in good condition. The small deficit of 0.08 c.c. in nearly 60.00 c.c. may be due to some slight leaks on the lower section which were not discovered, or it may be due to unavoidable errors in the work.

The tests of this series will appear to be more harmonious when compared with the Holyoke tests by using Plate LI.

Tests 8, 16, 17, and 18 are very doubtful as plotted. They were off-speed tests, but the principal cause of deviation from correct plotting is probably due to lack of data as to distribution of salt and relative weights. This applies to all the partial tests.

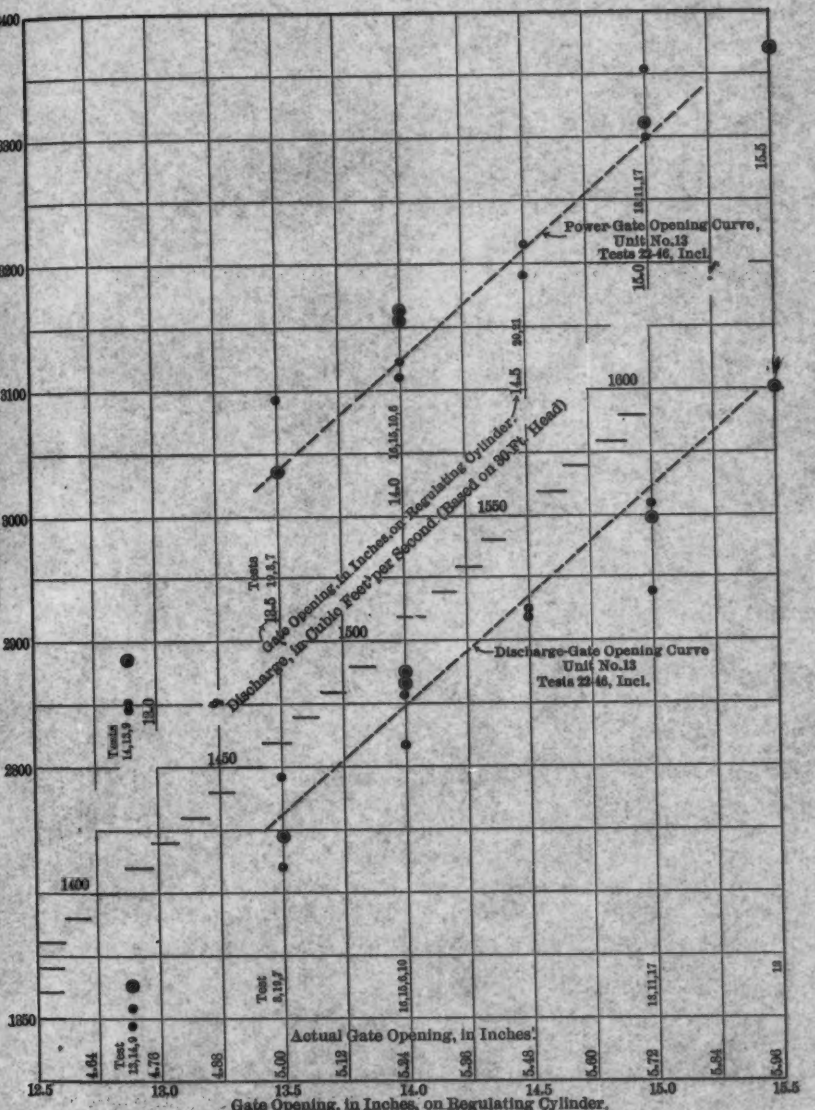
Plate XLVII, Unit No. 13, Second Series, Second Plotting.—After plotting this series, as on Plate XLVI, where certain tail-water samples were eliminated owing to supposed leaks, the tests were re-plotted, taking into account all the titrations as usual, without regard to the possibility of leaks. This plotting is shown on Plate XLVII, and it will be seen that the changes in main are small, but that the final plotting checks that of the third series even better than that of Plate XLVI.

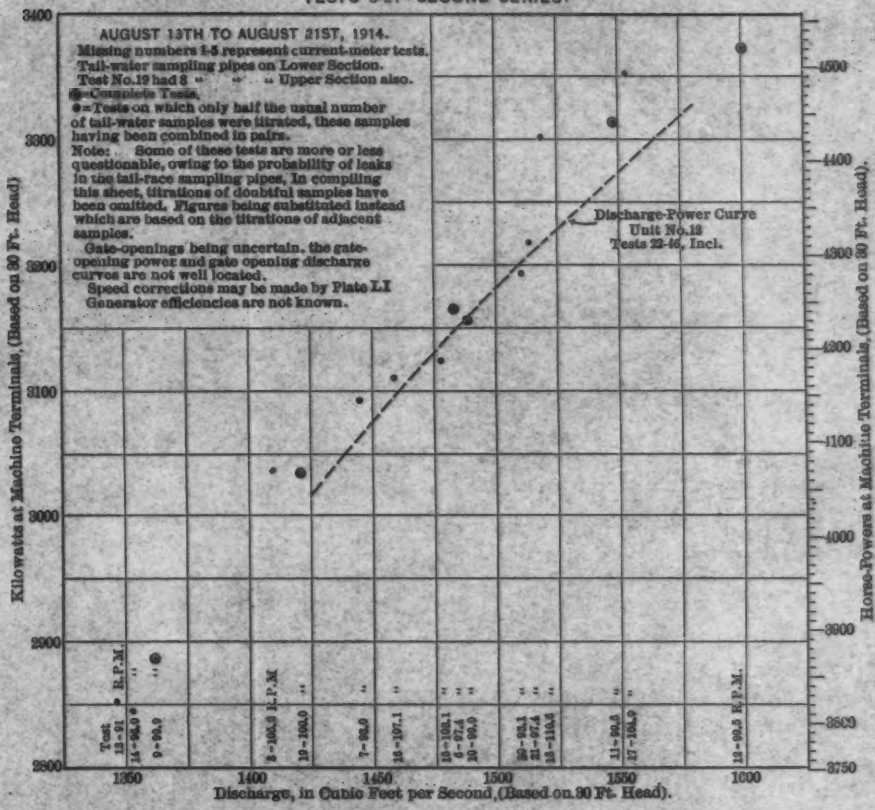
Plate XLVIII, Unit No. 13, Third Series.—This series is remarkable for its self-consistency and for the high degree of precision attained in so far as accidental errors are concerned. It has also been shown in Part II that there cannot be any serious systematic error in this series. In all probability, they are the most precise discharge measurements ever made on a hydro-electric unit of large capacity for water.

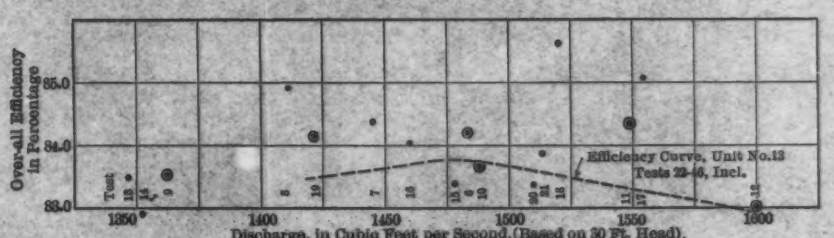
Test 39 is the only one about which there is any question. Unfortunately, during this test, the greater portion of the head-water observations was lost, thus taking away any opportunity for arriving at the correct head based on observation. The head has been interpolated so as to harmonize the test with the others. Had these observations been lost on any other test, the matter would not have been so serious, because, in Test 39, the head varied by 1½ ft. or more, and it was desirable to have the results of a test, under such conditions, but the only chance in this series was lost.

The close agreement between concentrations on the upper and lower sampling sections in Tests 22 and 23 has been shown in Table 60 in connection with Plate XLVI.

FINAL CURVES.
TESTS OF UNIT NO.13
TESTS 6-21 - SECOND SERIES.







Gate Openings.—No effort was made, in any of the tests, to secure accurate records of gate opening. The gates were calipered in the usual manner, and a curve connecting the actual opening with gate-register indications was drawn. These openings, as well as the corresponding gate-register readings, are plotted at the left of the plate, as in the preceding cases.

The method of setting the gate was always to open it from a relatively small opening to the one at which the test was to be run, thus avoiding the effects of back-lash as far as possible. It was quite possible, however, for the gate to move after being set, and it is very clear, from the diagram, that such motions did occur, and always in one and the same direction from the gate-power or gate-discharge curve. This would be the natural consequence of the method of setting the gate, and the consistency of the plottings in this respect adds materially to the weight of the statements concerning the accuracy of this series of tests.

The real precision of the tests comes into view when power and discharge are plotted against one another. An extremely accurate efficiency curve results by computation from this discharge-power curve, and it is seen that, of the plottings of the efficiencies from Table 56, only two or three of the thirteen fall away from the curve by as much as 0.1 per cent.

Plate XLIX, Unit No. 12.—This is the only series of tests on Unit No. 12. The plottings were made from the data of Table 57. Tests 209 and 211 are the only ones plotting seriously wild. Test 211 seems to be low on power, and 209 is high on discharge. Otherwise the tests leave no doubt as to the fact that Unit No. 12 is not so efficient as Unit No. 13. The fact that accidental errors are larger than for Unit No. 13 is due to the greater haste with which the tests were made and to several changes in the personnel of observers, the new men not being so well trained in their work. This is especially true of the power, which, however, is quite consistent with the Allis-Chalmers readings, excepting for Tests 211 and 212. In these two the Allis-Chalmers instruments are inconsistent with both the standard and the switch-board instruments, as the ratios in Table 61, computed by slide-rule, indicate.

The Leeds and Northrup and switch-board readings being the more consistent, it would scarcely seem that the power can be seriously in

error. It is more probable that the wild plottings result from an accumulation of small errors due to one cause or another. A further adjustment of the tests might be made, but this would not make any material change commensurate with the work necessary to effect a satisfactory adjustment.

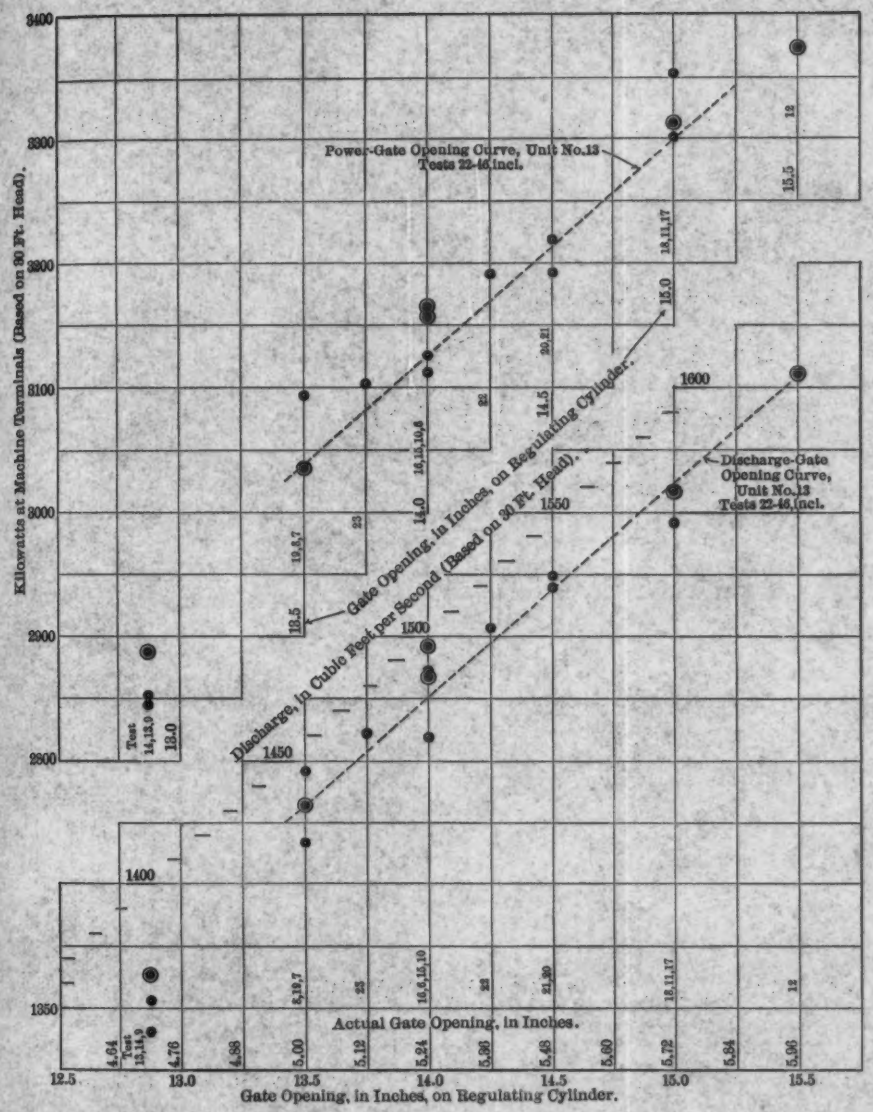
TABLE 61.—COMPARISON OF THE RATIOS OF POWER READINGS FOR THE SWITCH-BOARD, ALLIS-CHALMERS, AND LEEDS AND NORTHRUP, DIRECT-CURRENT INSTRUMENTS IN THE TESTS ON UNIT NO. 12.

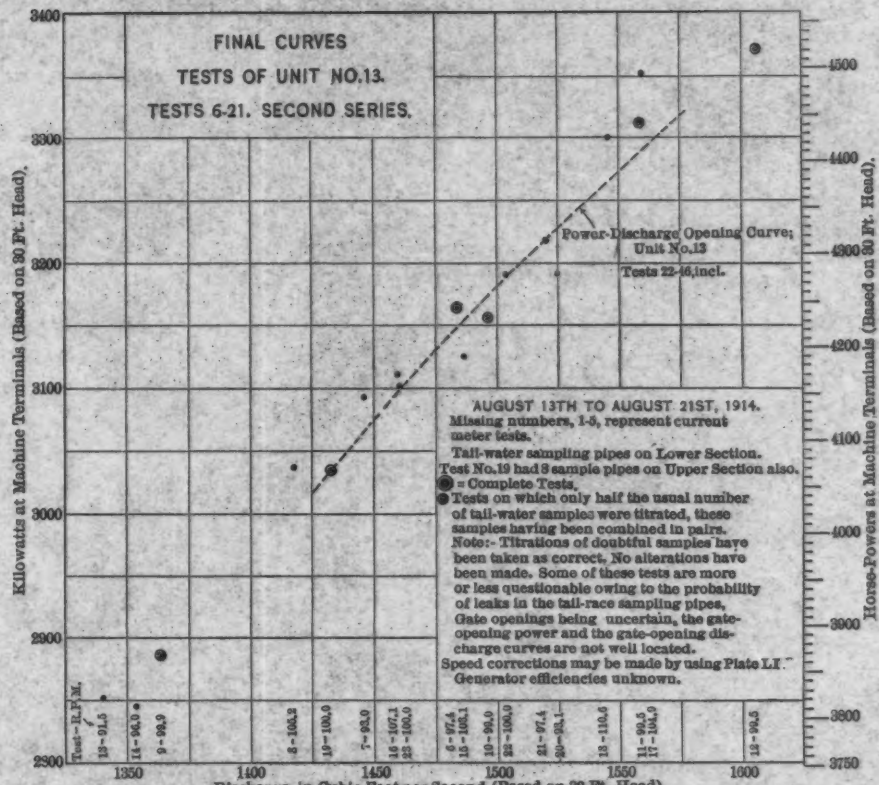
Test No.	Leeds and Northrup / Switch-board	Leeds and Northrup / Allis-Chalmers	Allis-Chalmers / Switch-board
200	1.016	1.001	1.014
201	1.021	1.001	1.020
202	1.026	1.000	1.026
203	1.019	1.003	1.019
204	1.023	1.002	1.021
205	1.030	1.004	1.026
206	1.023	1.002	1.021
207	1.028	1.002	1.026
208	1.030	0.999	1.031
209	1.027	1.004	1.026
210	1.030	1.003	1.026
211	1.021	0.989	1.033
212	1.025	0.991	1.034

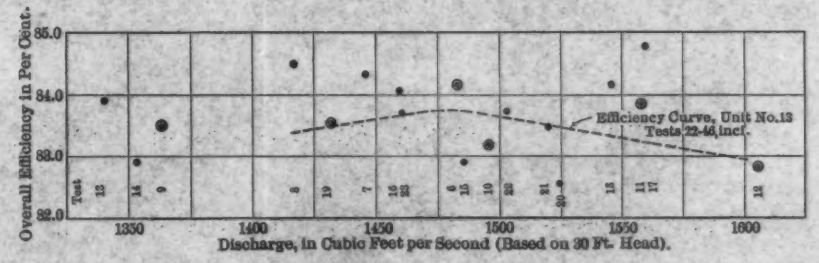
A more satisfactory procedure will be to select those tests which appear to possess the elements of precision. If several of the tests plot on both the gate-discharge and gate-power curves, and are otherwise consistent, they may well be given the greater weight in drawing the final efficiency curve, which, owing to the paucity of tests over a large range of gate openings for this unit, may be extended parallel to the efficiency curve of Unit No. 13, which has been drawn on the diagram as a broken line.

Tests 207 and 212 plot well in this respect, as do 200, 205, and 208. Several others, 202 and 210, for example, are not seriously off, in this regard, though a combination of discrepancies throws the former downward.

It is not difficult to account for a slight deficit in the performance of Unit No. 12 as compared with Unit No. 13. The hydraulic conditions were plainly not so good for the former. Indeed, the turbulence in the head-races of the units increases in violence as the west end of


Power-Gate Opening Curve, Unit No.13
Tests 22-46,incl.
Discharge-Gate Opening Curve, Unit No.13 Tests 22-46,incl.
Gate Opening, in Inches, on Regulating Cylinder.
Discharge, in Cubic Feet per Second (Based on 30 Ft. Head).
Kilowatts at Machine Terminals (Based on 30 Ft. Head).
Actual Gate Opening, in Inches.
Gate Opening, in Inches, on Regulating Cylinder.


FINAL CURVES
TESTS OF UNIT NO.13.
TESTS 6-21. SECOND SERIES.
Power-Discharge Opening Curve; Unit No.13
Tests 22-46,incl.
AUGUST 13TH TO AUGUST 21ST, 1914.
Missing numbers, 1-5, represent current meter tests.
Tail-water sampling pipes on Lower Section.
Test No.19 had 8 sample pipes on Upper Section also.
= Complete Tests.
Tests on which only half the usual number of tail-water samples were titrated, these samples having been combined in pairs.
Note:- Titrations of doubtful samples have been taken as correct. No alterations have been made. Some of these tests are more or less questionable owing to the probability of leaks in the tail-race sampling pipes. Gate openings being uncertain, the gate-opening power and the gate-opening discharge curves are not well located.
Speed corrections may be made by using Plate LI.
Generator efficiencies unknown.
Kilowatts at Machine Terminals (Based on 30 Ft. Head).
Horse-Powers at Machine Terminals (Based on 30 Ft. Head).
Discharge, in Cubic Feet per Second (Based on 30 Ft. Head).


Efficiency Curve, Unit No.13
Tests 22-46,incl.
Overall Efficiency in Per Cent.
Discharge, in Cubic Feet per Second (Based on 30 Ft. Head).

the power-house is approached, No. 11 being in a less favorable location than No. 12. There is also a possibility that No. 12 was not so well adjusted as No. 13, very small differences in this respect having an appreciable effect on the net performance of the unit. This, however, was contrary to the expectations of the makers, who claimed that Unit No. 12 was in better condition than Unit No. 13.

The probability of a constant error as large as 0.75% in the tests of No. 12 is very small, as has been amply shown by a full discussion of the errors of the chemical method in Part II. There may be a possibility of 0.1 or 0.2% error of this class, but, without further evidence, there is no justification for such an assumption.

Plate L, Unit No. 11.—The plotting of the first series of tests, 105-109, on this unit is shown by circles on Plate L. The discharge measurements are fairly consistent, but the power is not. Test 105 is wholly untrustworthy as to power, as the governor was free and the power absorbed by the water rheostat, which was improperly connected and hung in the tail-race, was affected by variations in water level and variations of salt concentration. Consequently, the gate varied throughout a large range, thus throwing considerable doubt on this particular test. All the other tests were made on the regular load, and an attempt was made to keep the power factor in the vicinity of 85. Table 62 gives the power factor as computed and also as read from the power-factor indicator.

TABLE 62.—Power Factor Observations in Tests on Unit No. 11. Second Series.

Test No.	Power factor computed.	Power factor indicated.
111	.....	.....
112	0.855	.....
113	0.814	.....
114	0.885	.....
115	0.851	0.845
116	0.883	0.868
117	0.862	0.950
118	0.843	0.857
119	0.867	0.920
120	0.865	0.950
121	0.852	0.860
122	0.864	0.860
Mean	0.849	0.889
Mean, excluding 112	0.853	

The average power factor, therefore, was almost exactly 0.85, which was the contract requirement.

All the tests of the second series have a very consistent relation to the Holyoke tests on the model runner when stepped up to the full size, as on Plate LI, with the exception of Test 122, which cannot be considered as trustworthy on account of the errors introduced into the tail-race samples by reason of a change to sample bottles which were not thoroughly cleaned, as mentioned in Section 109. Where there are divergencies from a satisfactory plotting on Plate L, there are also properly related divergencies of speed from the contract requirement of 100 rev. per min. On the whole, the second series of tests on Unit No. 11 approximates in precision the third series on Unit No. 13, with only such errors as may be expected with the less reliable power readings and the greater haste in making the tests on the former. The chemical work is not quite as satisfactory as on Unit No. 13, but shows no serious evidence of rough work.

The fact that the efficiency does not show as high, relatively, as that of the direct-current machines is probably due to the fact (mentioned in connection with the tests on Unit No. 12) that the hydraulic conditions grow steadily worse as this end of the power-house is approached from the east, where Units Nos. 13, 14, and 15 are situated. On the whole, there is no evidence that there is any serious constant error in respect to the finally determined efficiency curve for Unit No. 11.

111.—Diagram of Turbine Performance.—It is only very recently that systematic attempts have been made to secure greater efficiency of power-house operation. This is in line with suggestions made in reports by the writer in 1910 and subsequently. As a general rule, each power development will require individual investigation to secure a complete answer to this question. There is one feature of it, however, which can be treated generally, and that is the characteristics of the hydro-electric units in the power-house. The performance of these units is subject to exact analysis, and diagrams of performance can be constructed which will facilitate an intelligent and effective oversight of this operation.

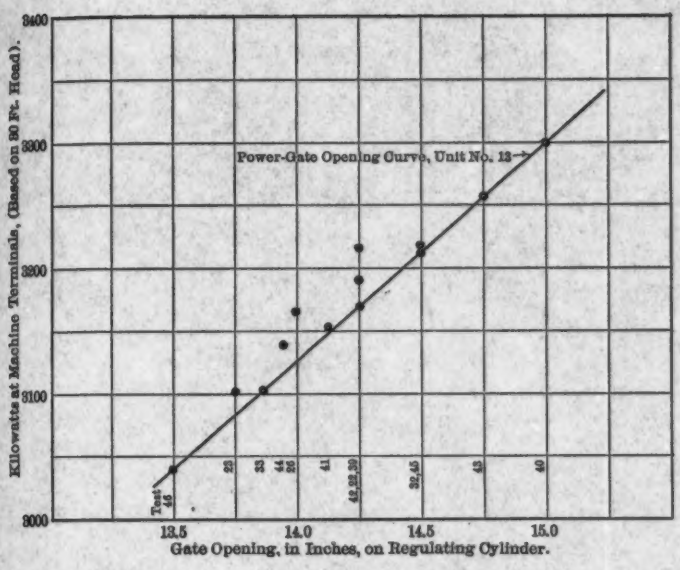
Most of the diagrams which have been constructed for the analysis of turbine performance have been made by designers and builders, and such diagrams, as a rule, are not adapted to the uses of power-house

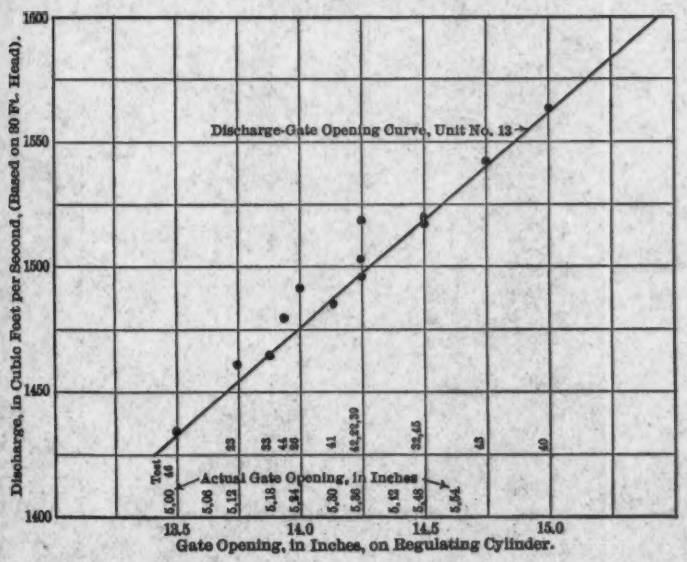
FINAL CURVES.
TESTS OF UNIT NO. 13.
TESTS 22-46. THIRD SERIES.
AUGUST 21ST TO SEPTEMBER 2D, 1914.

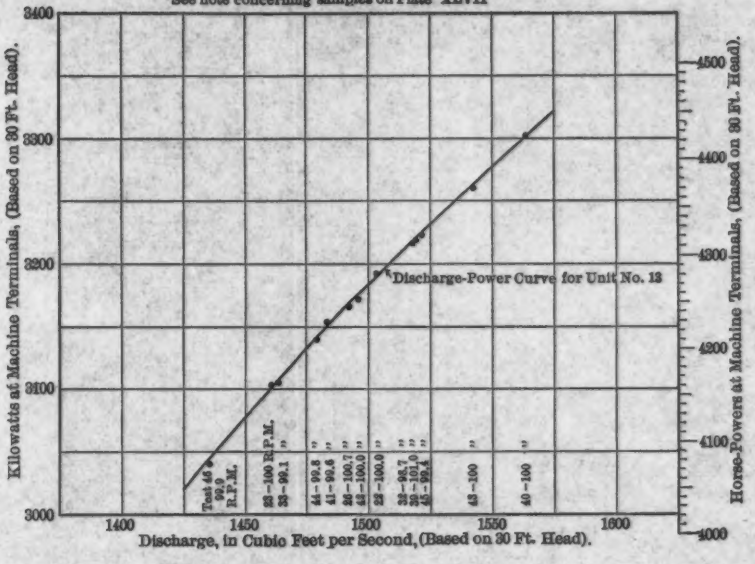
Tests 24, 25, 27-31, and 34-38 represent current-meter tests and are not shown on this plate. Dosed Tail Sampling Pipes on Upper Section, Tests 26-46. Dosed Tail Sampling Pipes on Lower Section, Tests 22-23. Tests 22 and 23 had 8 Sampling Pipes on Upper Section also. The speed in this series of tests is so near to 100 Rev. per Min. that no correction in plotting has been made, except in Tests 32 and 39. C—Plotting corrected to 100 Rev. per Min. by Plate LI. Gate openings being uncertain, the gate-opening power and gate-opening discharge curves are not well located.

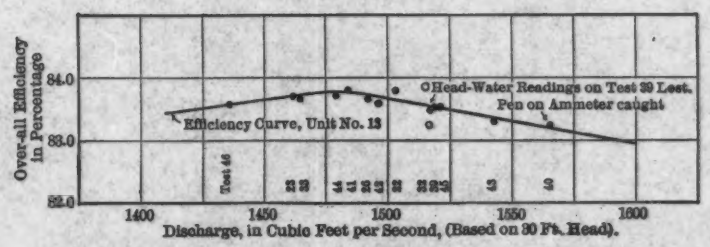
Generator efficiencies unknown. Test 19, Plate XLVII might be added to this series with propriety.

See note concerning samples on Plate XLVII









superintendents and overseers. Therefore, it is very desirable to have diagrams constructed especially for such purposes, and the following method has been used successfully by the writer for a number of years.

Relative to the performance of hydraulic turbines, theory and experience prove the following theorems, which, for brevity, are stated in the form of equations:

$$\text{At any fixed gate opening....} \left\{ \begin{array}{l} N = n\, h^{\frac{1}{2}} \\ P = p\, h^{\frac{3}{2}} \\ Q = q\, h^{\frac{1}{2}} \end{array} \right\} \text{.....(196)}$$

In which, N = number of revolutions per minute;

P = number of horse-powers developed by the runners;

Q = discharge, in cubic feet per second;

h = the head acting upon the wheels, in feet;

and n, p, q = obvious coefficients of performance.

These coefficients have been defined by some writers as being, respectively, the speed, power, and discharge of the turbine when operating on 1 ft. head. That is, the corresponding values of N, P, and Q when $h = 1$ are the values of n, p, and q. This is somewhat misleading, because, under this extreme condition, it might be impossible to get the turbine to operate. At least, it is not likely that such a definition should be taken literally. Coefficient of performance is a better term.

Again, both theory and experience show that p and q depend on the value of n, so that, in addition to Equations (196), we have

$$\text{At any fixed gate opening....} \left\{ \begin{array}{l} p = f_1(n) \\ q = f_2(n) \end{array} \right\} \text{.....(197)}$$

which symbolize the fact that both p and q are determined as soon as the value of n becomes fixed. Therefore, when n is fixed, both P and Q are definitely connected with h. In words, the theorem may be expressed as follows:

When the gate opening of the turbine and relative peripheral speed of the runner are held constant, that is, when the speed is adjusted to the head, the power of a turbine varies as the square root of the cube of the head, and the discharge varies as the square root of the head.

It is seldom, however, that the gate of a turbine can be set accurately by the ordinary gate mechanism, and, conversely, it is seldom that

the opening can be determined from the reading of a register operated by the mechanism. In any case, it is not necessary to successful power-house operation to know at all times what the gate opening may happen to be. The object may be accomplished in another way. Instead of using gate opening, the discharge or power of the turbines, and therefore of the power-plant, much more important quantities for the power-house superintendent to know may be made use of with equal facility.

Thus the relations expressed in Equations (197), though they may not be stated in the form of equations, are, nevertheless, equivalent to two equations between the four quantities, n, p, q, and gate opening. Consequently, a single relation may be derived from them connecting only the three quantities, n, p, and q, the fourth having been eliminated. Therefore, it may be formally stated that:

Each of the three quantities, n, p, and q, are dependent on the values of the other two only, the relation being expressible in the form of an isogram.

In particular, p may be expressed as a curved scale (series of curves) drawn to the rectangular co-ordinates given by n as abscissas and q as ordinates without reference to gate opening.

Now, the efficiency of a turbine may be written

$$\phi = \frac{550}{\gamma} \frac{P}{Q\,h} \qquad \text{.....(198)}$$

from which, by substituting values from Equations (196), we have

$$\phi = \frac{550}{\gamma} \frac{p}{q} \qquad \text{.....(199)}$$

γ being the specific weight of water.

Therefore, ϕ, like p, may be represented isogrammatically with reference to the same system of co-ordinates, n and q.

It follows from these relations that a diagram can be constructed connecting the speed, head, and speed coefficient, n, according to the relation indicated by the first of Equations (196), resulting in an isogram giving the values of n in the form of a curved scale, where values of h and N are plotted as rectangular co-ordinates. Better still, the curved scale for the speed coefficient becomes a system of radiating straight lines if the square roots of h are taken as the ordinates, or abscissas, as convenience may suggest.

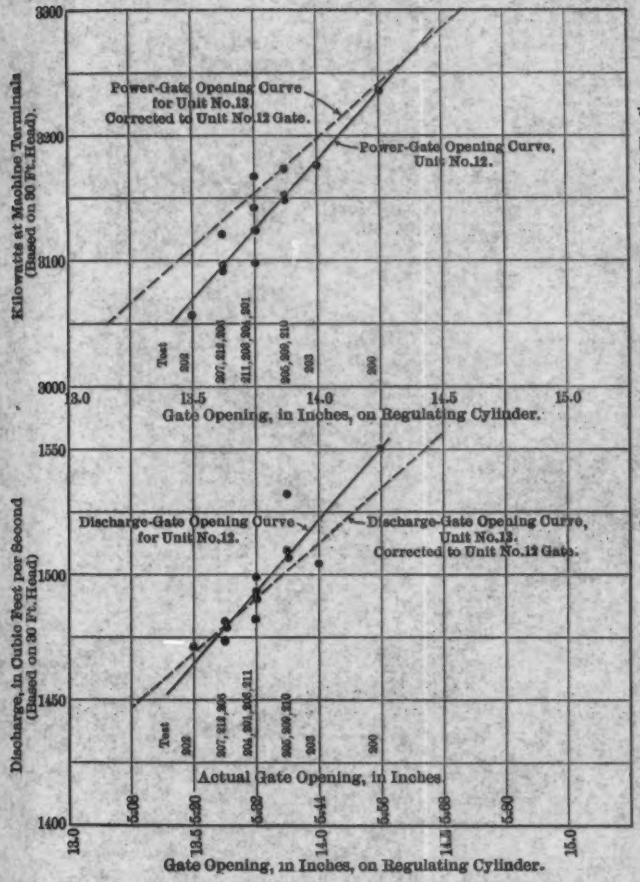
FINAL CURVES,
TESTS OF UNIT NO.12.
SEPTEMBER 28TH TO OCTOBER 13TH, 1914.

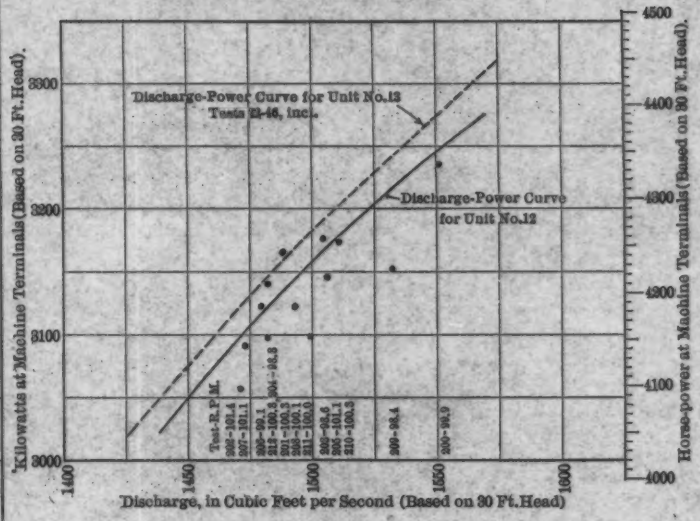
c=Plotting from tests corrected to 100 Rev. per Min. by Plate LI.

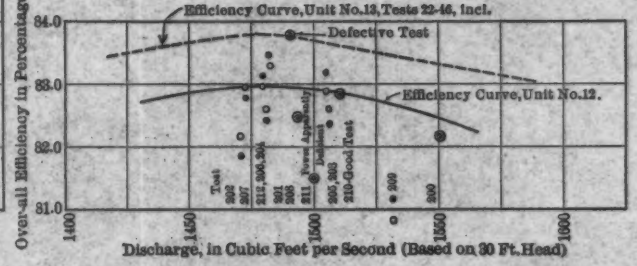
● Plotting from tests which do not change by Plate LI

Note: Correction of plottings by Plate LI uncertain to a small extent, as generator efficiencies are not known exactly.

Gate openings being uncertain, the gate-opening power and gate-opening discharge curves are not well located.







Hence this isogram for n, N, and h, when plotted to the square roots of h, provides a scale of straight lines representing n, and this scale may be taken as the abscissas of a system of co-ordinates in which an arbitrarily chosen scale for q represents the ordinates.

This latter system may then be used in constructing scales for ϕ and p, referred to n and q.

Finally, the original scale for head, h, may be used in conjunction with the scale for power coefficient, p, to form a system of co-ordinates (non-rectangular) for plotting the power of the turbine, P. In similar manner, the head scale and scale for discharge coefficient, q, determine a system of co-ordinates for plotting the discharge, Q, of the turbine.

Plate LI illustrates this method of plotting turbine performance. It gives the performance of one of the 80-in. twin-runner hydraulic turbine units which were tested, based on the results of Test No. 2 218 at Holyoke.

The speed and head scales and the sloping scale of relative speeds all appear as straight-line scales. These three scales and the corresponding quantities are connected by the first of Equations (196), the head scale being plotted to the square root of h and the speed scale being graduated according to the relative peripheral speed of the runner, rather than to n itself. This relation is given by the equation:

$$\text{Relative peripheral speed} = \frac{\pi}{60\sqrt{2g}} \frac{d\,N}{h^{\frac{1}{2}}} \qquad (200)$$

where d is the diameter (frequently only nominal) of the runner. If s is the relative peripheral speed, the relation expressed in Equation (200) may be written

$$s = 0.00653\, d\, n \qquad (201)$$

which is the equation used in the construction of the sloping scale of the diagram.

Having constructed the scale of relative speeds, or relative peripheral speeds, according to convenience, an arbitrarily chosen scale may be constructed for the discharge coefficient, q. The writer prefers to draw this scale to the left of the lowest number of revolutions per minute likely to be of frequent use. The scale may be composed of vertical parallel lines, conveniently spaced with regard to any desired degree of precision. The graduations for this scale appear along the upper margin of the diagram.

After constructing the discharge coefficient scale, which would be clearer if in a distinctive color, say green, the power coefficient may be plotted, say in carmine, with reference to n, or s, and q, according to the relations in Equations (196). Thus, from the records of the test,

$$\left.\begin{aligned} n &= \frac{N}{h^{\frac{1}{2}}} \\ p &= \frac{P}{h^{\frac{3}{2}}} \\ q &= \frac{Q}{h^{\frac{1}{2}}} \end{aligned}\right\} \quad \text{(See Equations (196))} \ldots\ldots\ldots (202)$$

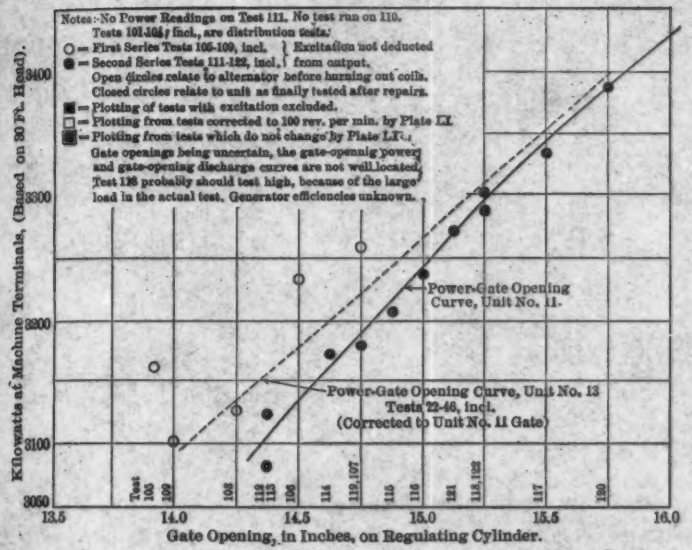
which furnishes the data for plotting p to the co-ordinates, n and q. If it is desired to plot to s and q, then Equations 200, or 201, must be used.

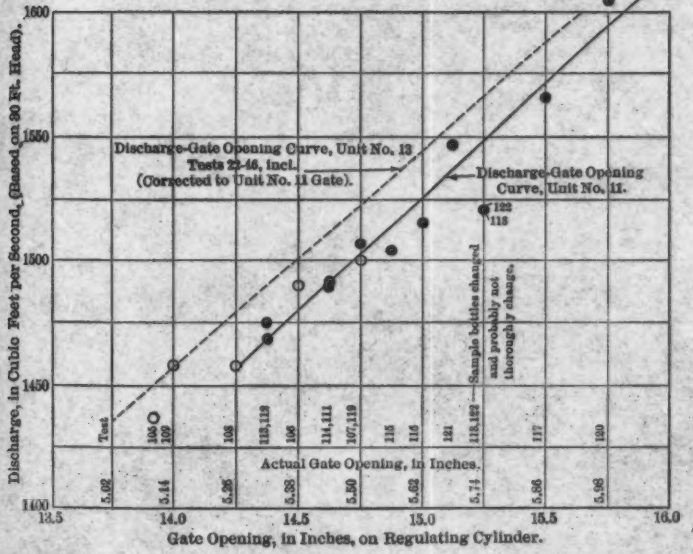
Turning attention now to the co-ordinate scales for p and q, it is seen that each intersects the original scale of heads, produced if necessary. Hence, by the second and third of Equations (196), scales for P and Q may be constructed, as shown on the left half of Plate LI. To be in keeping with p and q, the scales for P and Q should be in the same colors as their corresponding coefficients, p and q. Hence, in this case, the scale for Q may be green and that for P may be carmine.

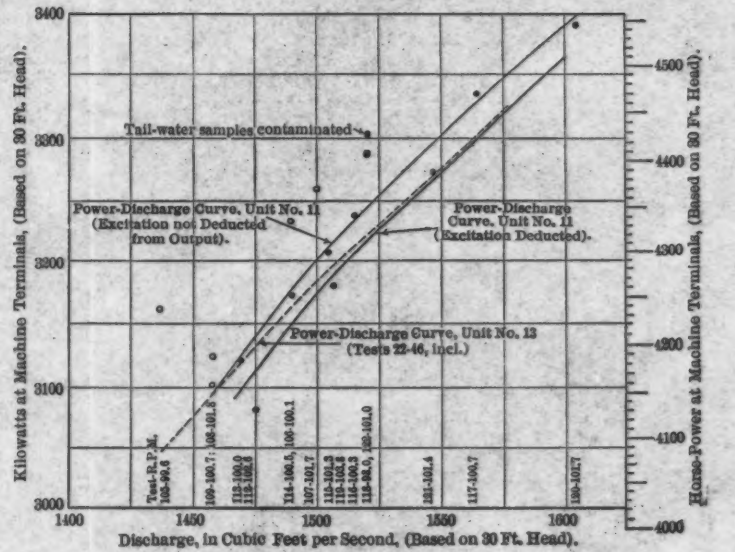
Finally, the efficiency curves may be plotted from the relations of Equations (198) or (199). These curves may be made more distinctive by being plotted in white on a dark background. If desired, curves of gate openings may be plotted in broken, or dotted lines, across the face of the diagram, thus making a complete picture of the operation of the turbine unit in its relation to the actual conditions of head, speed, power, discharge, efficiency, and gate opening.

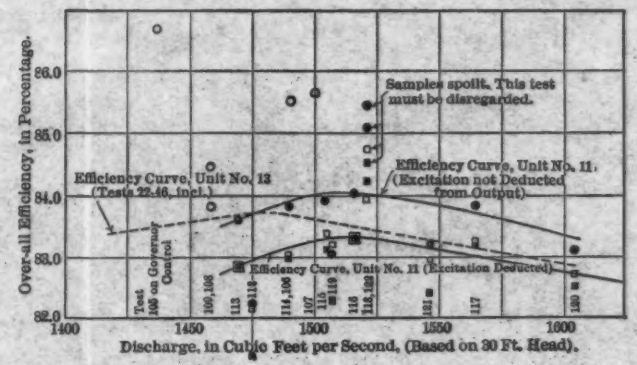
112.—Use of the Turbine Diagram.—Although the plotting of such a diagram involves considerable labor, its use is very simple, and the information it furnishes is extremely valuable in the supervision of power-house operation. The superintendent is able, in a few moments, by examining the diagram, to tell whether a particular unit is operating under favorable conditions or otherwise. He can tell whether a unit is developing all the power it should, under the conditions. If the trash racks become choked, the loss of head makes itself felt by cutting down the output of the unit, and this is instantly shown by

FINAL CURVES.
TESTS OF UNIT NO. 11.
SEPTEMBER 12TH-SEPTEMBER 18TH 1914.
NOVEMBER 9TH-NOVEMBER 21TH 1914.









reference to the diagram with the actual power output, speed, and difference in elevation of head- and tail-water. An inspection of the racks is not nearly so effective as this comparison of the actual and diagrammatic outputs. The racks may be clear but the runners may be choked, perhaps not sufficiently to attract the attention of the operators by a large reduction of power, but the deficit will be clearly shown by the diagram. The superintendent can check the methods of the individual operators by examining the performance of the units at various times.

113.—To Find the Efficiency, Discharge, Relative Speed, and Gate Opening of the Unit, with Given Power, Speed, and Head.—Suppose the head is 30 ft., the speed is 100 rev. per min., and the power is 4 700 h.p. Enter the diagram on the right at 100 rev. per min. on 30 ft. head. The point thus located determines the relative peripheral speed to be about 79½%, which can be read at the extreme left of the diagram near the bottom. It is not necessary to make this reading unless desired. Fix the point just located on the diagram by making a dot, or conveniently with a thumb-tack, or pen-point laid on the diagram. Locate and fix another point at the intersection of 30 ft. head with the curved scale of power at 4 700 h.p. The intersection of the relative speed line through the first fixed point (interpolated by mental estimation when necessary) with the power coefficient line through the second fixed point (also interpolated in this case) locates a third point on the diagram, which, in this case, lies within the closed curve of highest efficiency, 90 per cent. This point should also be fixed on the diagram.

Vertically, above this last point, that is, parallel to the discharge coefficient lines, at the given head, 30 ft. in this case, locate a fourth point which determines the discharge, $Q = 1\,520$ cu. ft. per sec., as shown by the discharge scale.

The gate opening, according to the Holyoke test, can be determined from the gate-opening curves, dotted on the diagram, by making the reading at the third point located on the diagram, at the intersection of the relative speed and power coefficient lines, in this case within the closed curve of 90% efficiency, as has been explained. This is about 72% of full gate, according to the plotting. Gate openings are of little use for purposes of this kind, but discharges are very important. Hence it is preferable to have the discharge coefficients

plotted as parallel vertical lines, rather than the gate openings, as it is necessary to project the locations of points in the direction of the discharge coefficient lines very frequently, and these projections can be most easily and correctly made with reference to parallel straight lines.

In the supervision of power-house operation, it is recommended that frequent, daily, or even hourly, reports of the operating conditions be made. These reports should give the output of each unit, with its maximum possible output under the conditions. The actual total output of the power plant, with its maximum possible output under the conditions, should also appear. Where these actual and maximum possible outputs show unreasonable discrepancies, an investigation should be started at once to ascertain the cause.

114.—Proper Speed and Load for Turbines.—In the case of alternating-current machinery, the units are usually restricted to a certain definite speed. In this case the greatest of care should be exercised in selecting turbines which will drive the generators at that speed, simultaneously operating at their maximum efficiency and best rated load. If the intending purchaser will have the makers guarantee performance according to a diagrammatic plotting such as outlined, perhaps limiting the guaranty to a certain operating area, or areas, of the diagram, he can then assure himself in advance of the efficiency which he can obtain at his proposed plant. A contract which simply requires a certain maximum efficiency may fall far short of securing the desired results, for the reason that conditions may seldom admit of operation at that particular efficiency. The important relation of speed to power may be drawn off such a diagram very simply by first observing what relative speed line, or lines, pass through the most desirable field of conditions on the diagram. If the turbine can operate at its best load most of the time, the more restricted, smaller field of higher efficiencies can be made available; but, if the load must vary over a considerable range, it may be found that the higher efficiencies would seldom or never be available.

As an example of the relations between speed and load, Table 63 has been compiled from Plate LI, from which it appears that the turbine unit can develop 90% efficiency only between heads of 27.5 and 32.2 ft., if the speed must be held at 100 rev. per min. The best head is 30 ft., giving the widest range of power.

PLATE LI.
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CHEMI-HYDROMETRY.

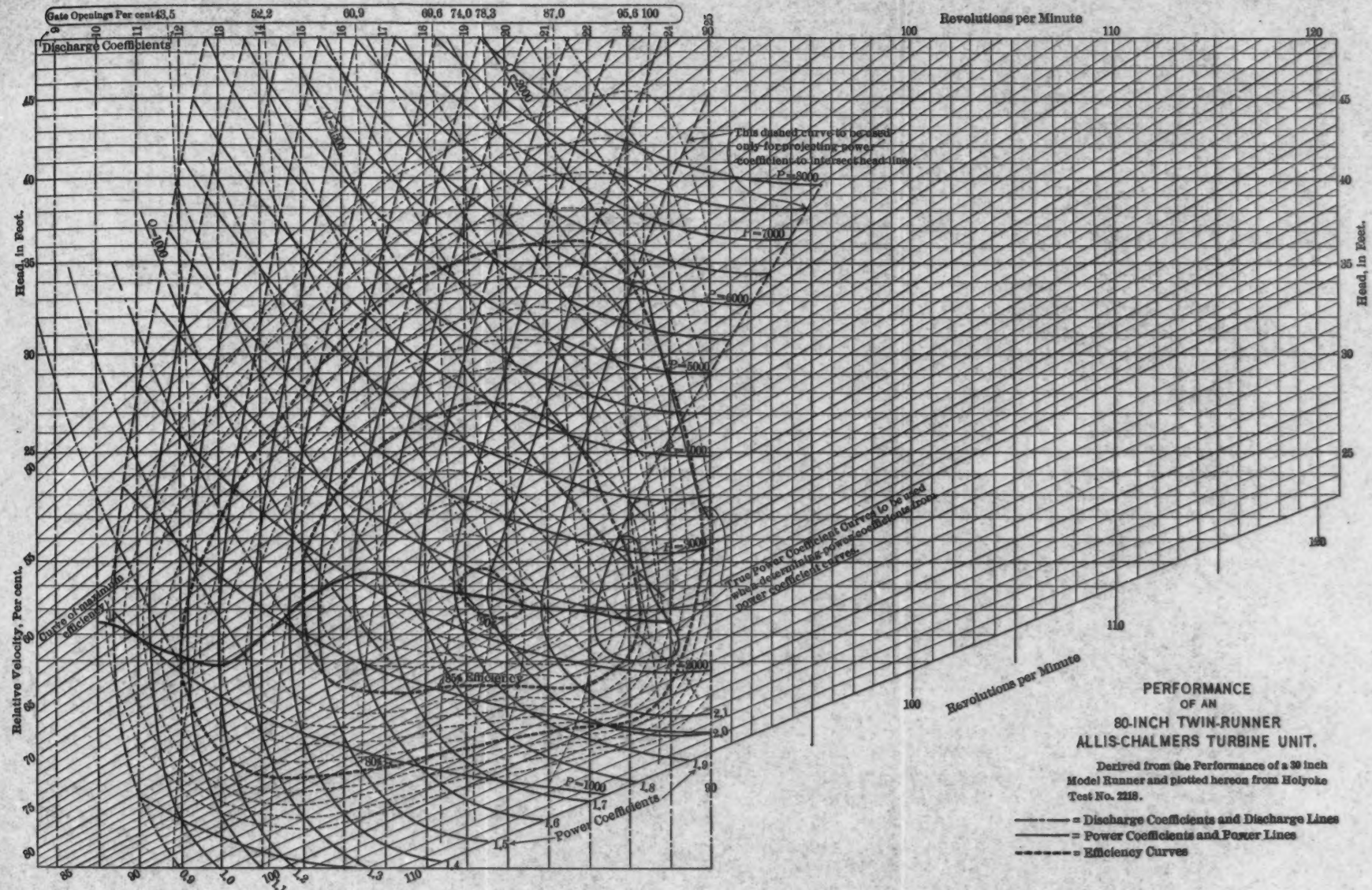

TABLE 63.—RANGE OF POWER AT VARIOUS HEADS DEVELOPABLE BY ONE TURBINE UNIT AT 100 REV. PER MIN. AND 90% EFFICIENCY, AS SHOWN BY PLATE LI.

Head, in feet.	Horse-Power.		Range of horse-power.
	Least.	Highest.	
27.5	4 300	4 300	0
28.0	4 400	4 500	100
29.0	4 470	4 660	190
30.0	4 600	4 900	300
31.0	4 820	5 050	230
32.0	5 100	5 200	100
32.2	5 200 +	5 200 +	0

Again, when wider ranges of power are required with this unit, it must be at the expense of efficiency. Suppose it is desired to ascertain the range of power at 88% efficiency. Such an investigation appears in Table 64, which shows that a small sacrifice in efficiency secures a very much wider range of operating conditions, which will be found very necessary in many cases. Contracts, therefore, should have due regard to such requirements of operating conditions, and diagrams similar to Plate LI will govern the specifications for turbine performance.

TABLE 64.—RANGE OF POWER FOR ONE UNIT AT 100 REV. PER MIN. AND 88% EFFICIENCY, AS SHOWN BY PLATE LI.

Head, in feet.	Horse-Power.		Range of horse-power.
	Least.	Highest.	
24.8	3 800*	3 800*	0
26.0	3 800	4 100*	300
27.0	3 900	4 300	400
28.0	4 000	4 600	600
29.0	4 100	4 800	700
30.0	4 200	5 000	800
31.0	4 400	5 200	800
32.0	4 550	5 400	850
33.0	4 750	5 600	850
34.0	4 900	5 800	900
35.0	5 100	5 950	850
36.0	5 300	6 150	850
37.0	5 500	6 300	800
38.0	5 800	6 300	500
39.0	6 200	6 200	0

*Somewhat doubtful.

In cases where the speed of the units may be varied within certain limits, as with direct-current generators, or isolated alternating machines, it may be desirable to adjust the speed to the head and power, so as to secure maximum efficiency. Table 65 is a study of this kind compiled from Plate LI.

TABLE 65.—ADJUSTING THE SPEED TO HEAD AND POWER WHEN FEASIBLE.

(The first item under any given power is the speed in revolutions per minute and the second item is the resulting maximum efficiency as given by Plate LI.)

Head, in feet.	Horse-Power.											
	3 500		4 000		4 500		5 000		5 500		6 000	
	Rev. per min.	Per-centage.	Rev. per min.	Per-centage.	Rev. per min.	Per-centage.	Rev. per min.	Per-centage.	Rev. per min.	Per-centage.	Rev. per min.	Per-centage.
25	89	90	98	88*	.....	.....	.....	.....	.....	.....	.....	.....
26	88	89	96½	90	.....	.....	.....	.....	.....	.....	.....	.....
27	87	88+	93	90+	.....	.....	.....	.....	.....	.....	.....	.....
28	87	87	93	89+	101	89+	.....	.....	.....	.....	.....	.....
29	88	86	92	88+	98	90+	105	88*	.....	.....	.....	.....
30	89	85	91	87+	97	89+	104	89+	.....	.....	.....	.....
31	91	84+	91	87	96	89	102	90+	108	88*+	.....	.....
32	94	83+	92	86	95	88	100	90+	108	89+	.....	.....
33	96	82+	94	85	94	87+	99	89	105	90+	111*	88
34	96	82	96	84+	95	86+	98	88+	103	90	111	89+
35	96	81	98	83+	97	85+	98	88	103	89	108	90+
36	96	80	100	83	98	85	98	87+	102	88+	106	90
37	96	79+	101	82	100	84	99	86	101	88	106	89
38	96	79	101	81+	103	83+	101	85+	100	87	105	88+
39	95	78	101	80+	104	83	102	85	102	86	104	88
40	95	77+	100	80	105	82	105	84	103	85	104	87+

* Doubtful if this could be fully realized.

In compiling a table of this kind, a line of maximum efficiencies for given power coefficients drawn on the diagram facilitates the compilation greatly. Thus, from the intersection of the given power and head, project the interpolated power-coefficient line by mental estimation until it intersects the curve of maximum efficiency. Enter in the table the efficiency reading from the efficiency scale. From the intersection last located and fixed, project the relative speed line by mental estimation until it intersects the given head line. Enter in the table the revolutions per minute, read from the speed scale, and proceed similarly with the next given head and power.

A similar table, taking into account generator losses and efficiencies, and thus relating to the performance of an entire unit, may be placed in the hands of the operator, with instructions to adjust the speed to the head and power according to the indications of the table.

Of course, a diagram based on the principles used in constructing the curves of Plate LI may be made, but taking into account, in addition, the performance characteristics of the generator, thus resulting in direct relations between the electrical output of the unit and the other quantities involved in Plate LI. For many purposes, such a diagram will prove to be of great value.

115.—Relations of Tests in Place to Holyoke Tests.—It is intended to make diagrams similar to Plate LI for the performance of the entire units, basing the data on the results of the tests, and taking into account, as fully as possible, the performance of the generators along with the turbines. Such a study will consume more time and space than the scope of the present paper will admit. Table 66, however, gives an approximate comparison of Holyoke tests with place tests based on assumed generator efficiencies. Owing to the uncertainty as to generator performance, no tests having been made on them, the comparison is only of passing interest.

It will be observed that there are relatively large efficiency corrections for Tests 32 and 39, Unit No. 13, due to relatively large deviations of the speed from 100 rev. per min. during the tests. It is doubtful if the speed was so much in error during these tests. Hence the corresponding discrepancies on Plate XLVIII of the corrected plottings of these tests, indicated by open circles, are, very likely, excessive.

Attention is called to this, as the fact suggests the desirability of having a complete count of the total number of revolutions during each test, along with frequent speed determinations for shorter intervals. Had this been done, it would be possible to locate the cause of the discrepancies in the two plottings.

TABLE 66.—Relations of Holyoke Tests to Tests in Place.
(Computed from Plate LI for 30 ft. head and 100 rev. per min.)

Test No.	Horse-power. Generator efficiency = 95 per cent.	Speed, in revolutions per minute.	Efficiency by test. Generator efficiency = 95 per cent.	Holyoke efficiency condition of test.	Comparison test efficiency to Holyoke efficiency.	Holyoke efficiency condition of 100 revolutions per minute.	Correction for plotting. efficiency.
Unit No. 11.							
112	4 310	102.6	85.8	87.4	— 1.6	88.1	+ 0.7
113	4 364	100.0	87.2	88.3	— 1.1	88.3	0
114	4 431	100.5	87.4	88.5	— 1.1	88.55	+ 0.05
119	4 446	103.8	86.6	87.7	— 1.1	88.6	+ 0.9
115	4 481	101.3	87.5	88.5	— 1.0	88.75	+ 0.25
116	4 529	100.3	87.7	89.0	— 1.3	89.0	0
121	4 572	101.4	86.8	88.9	— 2.1	89.4	+ 0.5
118	4 594	98.0	88.7	89.8	— 1.1	89.5	— 0.3
122	4 610	101.0	89.0	89.4	— 0.4	89.6	+ 0.2
117	4 668	100.7	87.6	89.9	— 2.3	90.0	+ 0.1
120	4 748	101.7	86.8	90.1	— 3.3	90.3	+ 0.2
Unit No. 12.							
Test No.	Horse-power. Generator efficiency. = 94 per cent.	Speed, in revolutions per minute.	Efficiency by test. Generator efficiency = 94 per cent.	Holyoke efficiency condition of test.	Comparison test efficiency to Holyoke efficiency.	Holyoke efficiency condition of 100 revolutions per minute.	Correction for plotting efficiency.
202	4 358	101.4	87.08	88.1	— 1.02	88.4	+ 0.3
206	4 450	99.1	88.42	88.8	— 0.38	88.6	— 0.2
207	4 409	101.1	87.99	88.3	— 0.31	88.5	+ 0.2
212	4 415	100.8	87.61	88.4	— 0.79	88.6	+ 0.2
201	4 517	100.3	89.10	88.9	— 0.20	88.9	0
204	4 479	98.8	88.82	88.9	— 0.08	88.7	— 0.2
208	4 453	100.1	87.67	88.7	— 1.03	88.7	0
211	4 418	100.0	86.63	88.6	— 1.97	88.6	0
205	4 489	101.1	87.61	88.6	— 0.99	88.8	+ 0.2
209	4 495	98.4	86.22	89.1	— 2.88	88.8	— 0.3
210	4 526	100.3	88.10	89.0	— 0.80	89.0	0
203	4 529	98.5	88.50	89.3	— 0.80	89.0	— 0.3
200	4 614	99.9	87.43	89.7	— 2.27	89.7	0
Unit No. 13.							
46	4 335	99.9	88.8+	88.2	+ 0.6	88.2	0
23	4 422	100.0	89.05	88.6	+ 0.45	88.6	0
33	4 425	99.1	89.0	88.7	+ 0.2	88.6	— 0.1
44	4 476	99.8	89.05	88.75	+ 0.30	88.75	0
41	4 496	99.6	89.12	88.9	+ 0.22	88.85	— 0.05
26	4 513	100.7	89.0	88.85	+ 0.15	88.9	+ 0.05
42	4 520	100.0	88.85	89.0	— 0.15	89.0	0
22	4 550	100.0	89.1	89.1	0.	89.1	0
32	4 580	98.7	88.8	89.6	— 0.8	89.3	— 0.3
39	4 586	101.0	88.8	89.1	— 0.3	89.4	+ 0.3
45	4 590	99.4	88.8	89.5	— 0.7	89.45	— 0.05
43	4 642	100.0	88.6	89.9	— 1.3	89.8	0
40	4 705	100.0	88.58	90.2	— 1.62	90.2	0

PART V.

CURRENT METERS.

SUMMARY.

116.—Current meters were run in parallel with the chemical equipment in a number of the earlier turbine tests as a precaution. These are marked: *D, E, G, H, I, K, L, N, O, S, T, U,* and *V.* Tests which were made with meters only are marked: *B, C, F, J, P, Q, R,* 1, 2, 3, 4, 5, 27, 28, 29, 30, 31, 34, 35, 36, 37, and 38. More than 4 000 individual velocity observations were made with the five current meters.

Ratings were made for these meters at the Naval Tank of the University of Michigan with the efficient equipment and expert advice of Professor H. C. Sadler. These ratings were made in still water, with and without oscillations. Oscillations of three kinds were caused: longitudinal, vertical transverse, and horizontal transverse, of various degrees of amplitude and period. Figs. 32, 34, and 36 are examples.

The particular meters rated were: Haskell No. 1, Haskell No. 2, Ott No. 2311, Ott No. 2312, and Price No. 1553.

The following propositions, which are restatements of those previously made by the writer* are verified, with some modifications in the case of the small Price meter, under certain circumstances.

When a cup meter is run in turbulent water it will register a larger number of revolutions per second than a perfect still-water rating would indicate.

When a screw meter is run in turbulent water it will register a smaller number of revolutions per second than a perfect still-water rating would indicate.

If both types of meter are used simultaneously in turbulent water, the disparity between the discrepant velocities thus determined by the still-water rating may be taken as a basis for correcting the discrepant velocities.

In these ratings, the tails, or rudders, of the meters were removed, and the meters were rigidly attached to oscillating arms capable of motions in the three directions mentioned. Three meters were thus rated simultaneously, the three arms oscillating in unison and the meters pointing in the direction of motion of the car.

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* *Transactions*, Am. Soc. C. E., Vol. LXXVI, p. 819.

This method of holding the meters rigidly in one direction is very unfavorable to the screw meter and favorable to the cup meter, the result being that the screws are retarded in most cases by a larger amount, and the cups are accelerated by a smaller amount, than would be the case were the meters free to swing.

It appears to be the case that, for velocities greater than 1.5 ft. per sec., the relative errors of all types of meters are nearly, if not quite, constant for the same relative amount of oscillation or turbulence. This further appears to be nearly realized for velocities of less than 1.5 ft. per sec. Thus, in the case of Ott meter No. 2311, Fig. 34, the retardation is uniformly 5 to 6% for transverse oscillations averaging about 30% of the velocity of the rating car, with amplitudes of 2 or 3 ft.

In the head-races during the turbine tests two Haskell meters, two Ott meters, and one Price meter were attached to a rack, composed of angle irons, Fig. 25, Section 97, in exactly the same manner as they were attached to the rating car, the eddying motion of the water, however, taking the place of oscillations.

It was shown by Tests 24, 25, 70, 71, 72, and 73 that this rack had a retarding influence on the meters of a greater or less degree. It is clear that the rack would cut off the water flow in the obstructed part of the section, crowding it into the rest of the raceway. Thus a meter in front of the rack would record lower velocity and one in the unobstructed part of the section higher velocity than with the rack removed.

In the vertical transverse oscillating tests of the small Price meter, there are certain relative amounts of oscillation which have the effect of retarding the meter rather than accelerating it. Above and below this relative amount, acceleration results. To this extent the cups are retarded rather than accelerated, but, on the average, as in the usual gauging section, and as in the case of the present tests, it appears that the net effect on the meter was acceleration.

Longitudinal oscillations have very little effect on the still-water ratings of the meters. For this reason the longitudinal oscillations are neglected in the discharge computations.

117.—Current-Meter Tests.—It would not be rational to conclude this paper without making a comparison of the results by the chemical

method with those by current meters. It is of very great interest to learn how closely the two methods consist with each other. This comparison has a double value. It confirms the theories advanced for chemi-hydrometry and simultaneously shows how accurate current-meter work can be, under the very unfavorable conditions of flow in a head-race where no precautions have been taken for "stilling" the water for the purpose of gauging the discharge.

Although the two methods check each other well within 1%, it would scarcely be wise to assert that current meters, under these adverse conditions, will always yield results correct to a fraction of 1 per cent. It is clear, however, that, if meters of properly related types are used in conjunction with an intelligent statistical study of their performance, results of a remarkably high degree of precision can be obtained, even under very unfavorable hydraulic conditions.

No attempt will be made to effect a complete digest of all the current-meter observations. The object will not be to make the best possible use of the current-meter data, but rather to arrive at a hasty estimate of the discharge for each test where the current meters and chemicals were used in parallel, thus leading to an immediate comparison of the two methods.

118.—Assumptions.—It is always necessary to make assumptions when computing rates of discharge based on the records of current-meter observations. The usual method requires the assumption that the meters perform the same in turbulent water as in still water. This assumption is entirely too narrow for precise work, and has not only been controverted on many occasions by eminent engineers, but has been proved beyond question to leave the resulting estimates in doubt by a large margin.

The results will be the more reliable the more nearly the assumptions are in accord with the facts. In this series of tests, therefore, it is assumed that both horizontal transverse and vertical transverse oscillations of the meters in the ratings represent horizontal and vertical disturbances in the meter section during the actual discharge observations. The effects of longitudinal oscillations have been shown in the ratings to be very small. It is assumed as a consequence that the effects of longitudinal perturbations, or pulsations, in the meter section are also very small. Such effects, accordingly, are neglected.

The ratings at Ann Arbor show conclusively that, for the method of holding the meters, the small Price meter will, on the average, be accelerated, and the Haskell and Ott meters will be retarded, and that the Haskell meter will be retarded by a larger amount relatively than the Ott meter. These differences in the characteristics of these three types make determinate the problem of estimating the discharge with a considerable degree of precision. Two meters which perform exactly alike, thereby giving perfectly consistent results, however, are worthless for the purpose of measuring velocities in turbulent water with precision.

The fact, stated by W. G. Price, M. Am. Soc. C. E., in his discussion of the writer's former paper, that vertical disturbances (when the meter axis is vertical) tend to counteract the effects of horizontal components of motion, does not assist in determining true velocities by using two or more different types of meter, but, on the contrary, rather hinders the determination. This hindrance arises from the necessity of making diagrams for the purpose of separating the two classes of disturbances and, indeed, of determining, so to speak, the relative amounts of horizontal and vertical disturbance.

In this respect it would facilitate matters considerably to have meters each of which suffers equal deviations for equal degrees of either of these two classes of turbulence. The Haskell and Ott meters are of this kind, but both are retarded, whereas, for the most accurate work, the deviations should be in opposite directions. In the present case we must rely principally on the true velocities as determined by the two combinations, Haskell-Price and Ott-Price, though the combination, Haskell-Ott, must not be disregarded.

It may be repeated here that, by properly designing a runner of the screw type with the vanes slightly cupped in the right sense, it should be possible to construct a current meter giving records practically correct in all usual cases of turbulent flow. Mr. F. Nagler has made such an instrument, though his experimental study is not yet completed. The last runner produced gives records which are more nearly correct than those of any meter with which the writer is conversant.

119.—The Statistical Method.—The term "statistical" as here used relates to the values of averages as deduced from aggregates rather than from the study of each individual element in detail. The usual

method in discharge gauging is to find a number representing the velocity at a particular point in the meter section, and to take the product of this velocity and the area associated with the point as representative of the discharge through this area. The sum of all such products is the discharge for the entire cross-section. In the statistical method, the aggregate, or sum, of all the velocities in the meter section may be multiplied by the distribution factor* to determine at once the mean velocity in the section. The area of the section, as a function of the elevation of the water surface, taken from a table, or curve, may then be multiplied by the mean velocity to determine the total discharge, thus saving a large amount of detail.

The distribution factor is the ratio of the mean velocity in the section to the sum of the metered velocities at a fixed set of points in the cross-section. If V_m is the mean velocity in the section (unknown), A the aggregate meter velocity, that is, simply the sum of the metered velocities, N the number of meter points, and v_m the arithmetical mean of the meter velocities, the distribution factor is:

$$\text{Distribution factor} = \frac{V_m}{A} = \frac{V_m}{N\,v_m} \quad \ldots\ldots\ldots\ldots (203)$$

Now, N is known, and the ratio, $V_m \div v_m$, is confined within very narrow limits for a large variety of cases if the meter points are distributed uniformly, or nearly uniformly, over the area. When the meter points are spaced uniformly across the usual river, the ratio is decreased to a somewhat larger extent, but even then it is pretty well confined to values between 0.92 and 0.96 for such sections as are chosen for gauging stations. For a liberal number of points distributed uniformly over the area of a large raceway, even when the conditions of flow are unfavorable, the ratio usually lies between 0.98 and 0.99, 0.985 being a close average. The tendency is for this value to increase with the size of the section. There may be cases, even in small sections, where it would be larger than unity. If the meters can be placed at points which determine the mean velocities throughout equal portions of the cross-section, the ratio of mean velocity to mean metered velocity becomes equal to unity, and the distribution factor is then simply the reciprocal of the number of velocities, that is, of meter points.

* Defined in the paper on current meters previously referred to. *Transactions*, Am. Soc. C. E., Vol. LXXVI, p. 837.

The value of the ratio $V_m \div v_m$, during the tests of 1911, was found, by a careful Pitot tube survey, to be 0.981 for the tail-races of the turbines.

120.—Subdivisions of Raceway.—The method of suspending and operating the current meters has been described fairly well in Part III. The equipment was arranged so that, by a simple adjustment, the meter points were placed at the centers of equal areas. The raceway was divided into 10 equal vertical and 10 equal horizontal strips, the 100 meter points being determined by the intersections of the 10 medians of the horizontal strips with the 10 medians of the vertical strips. A diagrammatic scheme is given herewith which represents the subdivisions of the raceway.

DIAGRAMMATIC SCHEME FOR METER POSITIONS.

Horizontals.	Depths.	POSITIONS OF METERS ON RACK AND POSITIONS OF RACK.									
		1		2		3		4		5	
		W.	E.	W.	E.	W.	E.	W.	E.	W.	E.
1	1										
	2										
2	3										
	4										
3	5										
	6										
4	7										
	8										
5	9										
	10										

The rack carrying the five current meters could be lowered until the meters were at the level of any set of meter points. By shifting the movable leaf of the rack until it was brought into contact with the west wall of the race, the five meters took their respective westerly positions for the set of meter points. After the velocities were determined at these points, the rack was shifted to its easterly position, and the remaining five velocities for this level of the rack were determined. The rack was then lowered to the next level.

In order to reduce the labor of computation, the five positions and five horizontals, respectively located by consecutive pairs of verticals and horizontals, will be used, instead of the individual meter points. Thus the square representing the area for the easterly and westerly situations of the rack for any position (Position 2, for example) in combination with any horizontal (Horizontal 3, for example) contains four meter points, and the sum of the corresponding velocities will be frequently treated in the sequel rather than the velocity at any one of the four points. This means a reduction in subdivisions from 100 to 25.

It will be observed that any one traverse of the raceway enables any one meter to determine the twenty velocities in the vertical which are determined by the position of the meter on the rack. This vertical, or position, may be treated as one of the five equal parts into which the race is divided, and the sum of the twenty corresponding velocities may be used as an aggregate, thus reducing the number of equal subdivisions to 5 instead of 25.

The necessity for these subdivisions is brought about by the fact that the meters do not give true velocities. Otherwise the aggregate for the entire cross-section would be immediately available for the determination of discharge.

121.—Current-Meter Records.—Tests G, H, I, K, L, N, O, P, Q, R, S, T, U, and V.—Instead of giving the current-meter records in full, a somewhat condensed form will be adopted for the purpose of simplifying the calculations. There were 100 velocity readings in each of the tests excepting those in which two traverses of the head-race were made by the meters, in which case 200 velocity readings were made. Test *O* is the only one of the latter description. In this test the average of the two readings at any one point is treated exactly as were the single readings in the other tests.

Two readings are given by any meter at each depth, one for the easterly and one for the westerly position of the rack. The race is divided into five equal strips, each containing two depths at which the meters were read. These strips will be called "horizontals". Thus, for any one position and any one horizontal, there are four velocity readings, and it is the sum of these four readings which appears in the following assemblage of data under any position and opposite any horizontal. Decimal points have been omitted, except in the means for

each meter and position, at the foot of each column. For example, the sum of the four velocities in Test *G* by Haskell meter No. 1, in Position 2, Horizontal 3, is 1 030, or 10.3 ft. per sec. The sum of all the velocities in Position 2 by this meter is 4 949, or 49.49 ft. per sec., and the mean velocity is 2.474 ft. per sec.

The writer is indebted to H. W. King, M. Am. Soc. C. E., for the use of a small Price meter and to Mr. W. M. White for the use of two Ott meters. These, with the two large Haskell instruments, were used in the Ann Arbor ratings, and in these tests.

Tables of Tests *G, H, I, K, L, N, O, P, Q, R, S, T, U* and *V*, are given showing the sums of the four meter velocities for each of the twenty-five equal rectangles formed by the lapping of the five vertical rectangles representing the meter positions, with the five horizontal rectangles representing the five pairs of consecutive depths at which the meters were operated.

The numbers in the tables are in units of $\frac{1}{1000}$ ft. per sec., decimal points having been omitted. The means are in feet per second.

TEST *G*.—JULY 22D, 1914.

Horizontal.	Position and Meter.				
	1	2	3	4	5
	O_1	H_1	P	H_2	O_2
1	1 098	1 033	1 038	1 015	1 068
2	1 227	1 051	1 058	959	1 020
3	1 238	1 030	1 033	986	1 034
4	1 071	1 011	992	982	1 186
5	828	824	946	809	720
Totals	5 462	4 949	5 067	4 751	5 028
Means	2.731	2.474	2.534	2.376	2.514

122.—Rating Current Meters.—In order to secure accurate results by current meters in turbulent water, it is necessary to use more than one type of meter, in the absence of an instrument which will not deviate from its still-water rating. Accordingly, three types were used in the tests: two of the Ott, two of the large-sized Haskell, and one of the small Price type. The letters, H, P, and O, will indicate the

corresponding types, and subscripts, where necessary, will indicate the particular number of the instrument referred to, according to the following scheme:

H_1 = Haskell meter No. 1,
H_2 = Haskell meter No. 2,
O_1 = Ott meter No. 2111,
O_2 = Ott meter No. 2112,
P = Price meter No. 1553.

TEST *H*.—JULY 24TH, 1914.

Horizontal.	Position and Meter.				
	1	2	3	4	5
	O_2	O_1	H_1	P	H_2
1	1 162	1 032	980	1 105	1 088
2	1 166	1 041	1 041	1 114	1 315
3	1 120	1 011	1 060	1 137	1 173
4	1 076	1 000	881	1 161	587
5	886	855	699	1 134	512
Totals	5 410	4 939	4 661	5 651	4 675
Means	2.705	2.470	2.330	2.826	2.338

TEST *I*.—JULY 24TH, 1914.

Horizontal.	Position and Meter.				
	1	2	3	4	5
	H_2	O_2	O_1	H_1	P
1	1 058	987	1 000	1 061	1 368
2	1 221	1 051	1 001	1 062	1 252
3	1 118	997	1 024	1 083	1 318
4	1 061	1 006	951	941	975
5	852	784	755	761	852
Totals	5 310	4 825	4 731	4 908	5 765
Means	2.655	2.412	2.366	2.454	2.882

TEST *K*.—JULY 24TH, 1914.

Horizontal.	Position and Meter.				
	1	2	3	4	5
	P	H_2	O_2	O_1	H_1
1	1 149	1 038	1 037	1 094	1 009
2	1 242	1 037	1 046	1 058	1 014
3	1 268	1 011	1 004	1 029	1 128
4	1 056	988	926	1 048	736
5	936	794	706	816	465
Totals	5 651	4 868	4 719	5 045	4 352
Means	2.826	2.434	2.360	2.522	2.176

TEST *L*.—JULY 24TH, 1914.

Horizontal.	Position and Meter.				
	1	2	3	4	5
	H_1	P	H_2	O_2	O_1
1	1 117	1 105	984	1 017	1 030
2	1 120	1 152	1 025	1 060	1 086
3	1 219	1 077	1 004	988	1 167
4	1 111	961	925	989	1 121
5	793	912	707	833	683
Totals	5 360	5 207	4 645	4 887	5 087
Means	2.680	2.604	2.322	2.444	2.544

Three meters could be rated simultaneously, with or without oscillations. Each instrument was attached to an arm at the rear of the rating car, and all three arms were tied together in such a manner that any oscillation imparted to any one meter would cause all three to oscillate in unison over equal amplitudes and in the same direction. In this way the separate effects of equal oscillations on all three meters could be recorded.

TEST *N*.—JULY 28TH, 1914.

Horizontal.	Position and Meter.				
	1	2	3	4	5
	O_1	H_1	P	O_2	H_2
1	1 358	1 135	1 185	1 102	1 073
2	1 203	1 125	1 220	1 126	1 112
3	1 301	1 106	1 245	1 129	1 066
4	1 104	1 120	1 154	1 072	1 065
5	995	878	1 013	912	841
Totals	5 961	5 364	5 817	5 341	5 157
Means	2.980	2.682	2.908	2.670	2.578

TEST *O*.—JULY 30TH, 1914.

(Two traverses were made of the meter section in this test. The numbers are averages for the two traverses.)

Horizontal.	Position and Meter.				
	1	2	3	4	5
	H_2	O_1	H_1	P	O_2
1	1 255	1 186	1 116	1 160	1 142
2	1 165	1 198	1 134	1 192	1 121
3	1 275	1 163	1 097	1 194	1 091
4	1 220	1 066	1 080	1 144	1 053
5	925	963	842	1 104	840
Totals	5 840	5 576	5 269	5 794	5 247
Means	2.920	2.788	2.634	2.897	2.624

Oscillations of various amplitudes and periods were tried, and in three directions: longitudinal, vertical, and horizontal transverse. The axes of the meters were parallel to the direction of the motion of the car in all cases, except that, in the longitudinal oscillations, the arms naturally gave the axes of the meters slight rocking motions in vertical planes parallel to the motion of the car. The effects of these longitudinal oscillations have been neglected, as they were not large.

TEST *P*.—JULY 30TH, 1914.

Horizontal.	Position and Meter.				
	1	2	3	4	5
	O_2	H_2	O_1	H_1	P
1	1 311	1 126	1 102	1 083	1 224
2	1 085	1 124	1 098	1 114	1 232
3	1 323	1 085	1 103	1 149	1 280
4	1 145	975	1 137	1 104	1 223
5	903	829	881	781	1 094
Totals	5 767	5 139	5 321	5 231	6 053
Means	2.884	2.570	2.660	2.616	3.026

TEST *Q*.—JULY 30TH, 1914.

Horizontal.	Position and Meter.				
	1	2	3	4	5
	P	O_2	H_2	O_1	H_1
1	1 518	1 175	1 123	1 099	1 161
2	1 243	1 188	1 179	1 099	1 101
3	1 361	1 143	1 059	1 117	1 092
4	1 216	1 089	1 019	1 107	1 067
5	1 001	926	772	1 010	847
Totals	6 339	5 521	5 152	5 432	5 268
Means	3.170	2.760	2.576	2.716	2.634

As a general rule, the meters were retarded by longitudinal oscillations for such velocities as we now have to measure, so that no surprise will be warranted if the final estimates of discharge prove to be somewhat deficient, as compared with results by more accurate methods than have yet been devised for current meters.

Figs. 32, 33, 34, 35, and 36 give, each on one sheet, the complete series of ratings for one or other of the five meters. The method of constructing the curves, being simple, requires no explanation.

TEST *R*.—JULY 30TH, 1914.

Horizontal.	Position and Meter.				
	1	2	3	4	5
	H_1	P	O_2	H_2	O_1
1	1 261	1 242	1 136	1 054	1 142
2	1 297	1 281	1 125	1 080	1 156
3	1 317	1 287	1 128	1 023	1 105
4	1 093	1 149	1 106	1 011	1 169
5	934	1 118	875	848	734
Totals	5 902	6 077	5 370	4 966	5 306
Means	2.951	3.038	2.685	2.483	2.653

TEST *S*.—AUGUST 8TH, 1914.

Horizontal.	Position and Meter.				
	1	2	3	4	5
	O_1	H_1	P	O_2	H_2
1	1 208	1 178	1 079	1 002	1 024
2	1 290	1 088	1 113	1 047	944
3	1 310	1 069	1 117	1 044	898
4	1 068	1 047	1 078	1 006	1 003
5	909	875	972	778	637
Totals	5 785	5 257	5 359	4 877	4 506
Means	2.892	2.628	2.680	2.438	2.253

123.—Composite Rating Curves.—Plates LII, LIII, LIV, LV, and LVI, are diagrams for determining the true velocity at a point by two velocity readings taken at this point by two meters of different type.

The true velocity may be read from the curved scale by locating a point on the diagram opposite the velocity, or revolutions per second, by one meter, and over the velocity, or revolutions per second, by the other meter, based on still-water ratings, thus representing the co-ordinates of the point on the diagram. The construction of the

TEST *T*.—AUGUST 10TH, 1914.

Horizontal.	Position and Meter.				
	1	2	3	4	5
	H_2	O_1	H_1	P	O_2
1	1 023	917	936	1 065	1 131
2	1 202	1 088	987	1 067	1 022
3	1 169	1 035	978	1 060	1 066
4	1 076	1 050	929	1 040	1 097
5	871	921	727	1 013	646
Totals	5 341	4 961	4 557	5 245	4 962
Means	2.670	2.480	2.278	2.622	2.481

TEST *U*.—AUGUST 10TH, 1914.

Horizontal.	Position and Meter.				
	1	2	3	4	5
	O_2	H_2	O_1	H_1	P
1	1 135	990	987	915	1 036
2	1 155	1 001	992	872	973
3	1 161	981	979	868	1 030
4	1 082	946	927	837	1 044
5	854	698	735	653	982
Totals	5 387	4 616	4 620	4 145	5 065
Means	2.694	2.308	2.310	2.072	2.532

diagrams from the ratings on Figs. 32, 34, and 36, will be sufficiently clear to the reader in principle, though, for accurate work, subsidiary diagrams segregating the ratings of different classes were made.

The curves on these subsidiary diagrams, when the ratings are not very complete, are not consistently spaced in all cases, and some minor adjustments were necessary to make them more harmonious and probably more nearly correct than they would have been otherwise.

It was never intended by the writer to make use of two meters which are each retarded, or each accelerated, to determine velocities in

PLATE LII.
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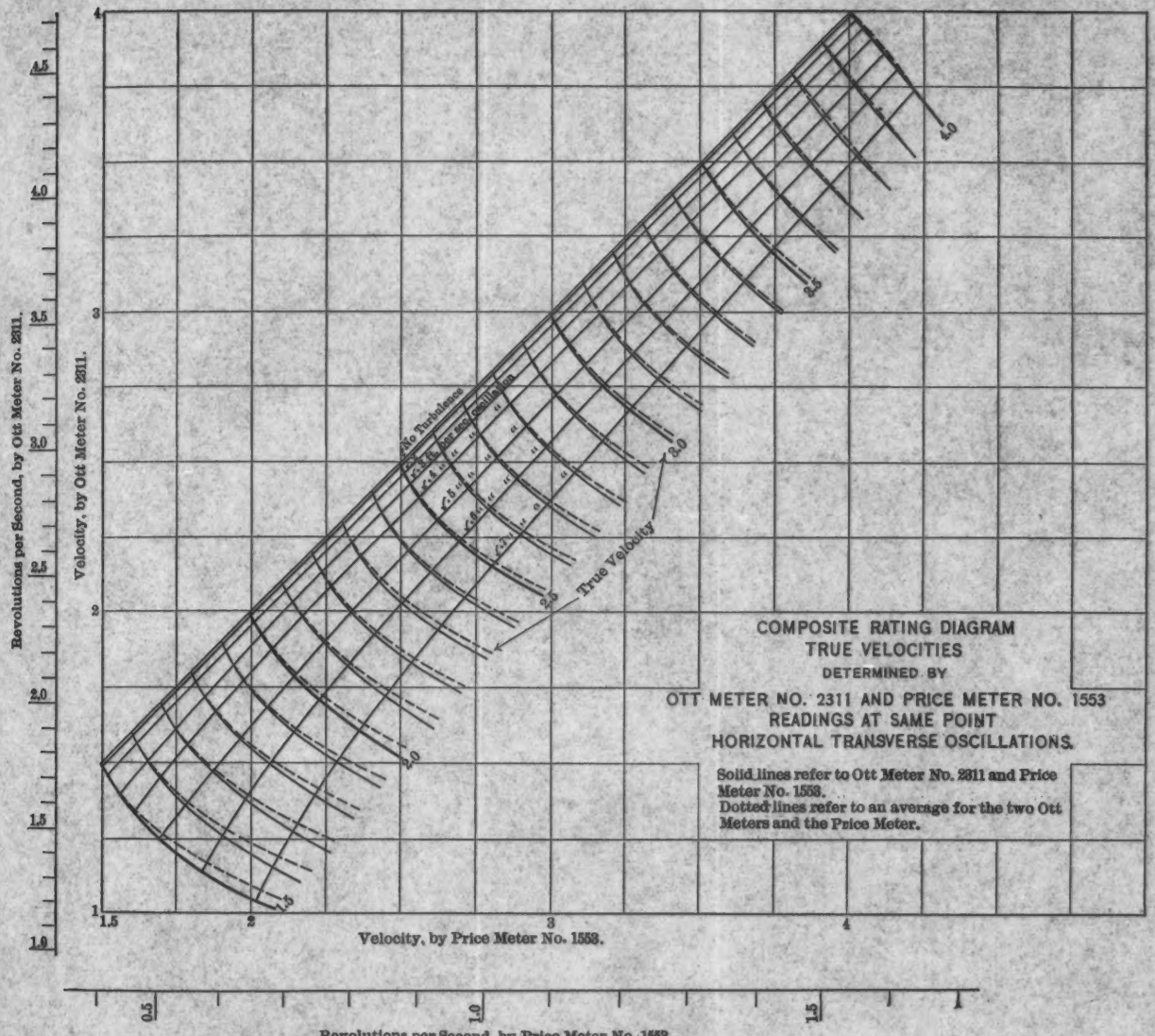
GROAT ON
CHEMI-HYDROMETRY.



COMPOSITE RATING DIAGRAM
TRUE VELOCITIES
DETERMINED BY
OTT METER NO. 2311 AND PRICE METER NO. 1553
READINGS AT SAME POINT
HORIZONTAL TRANSVERSE OSCILLATIONS.

Solid lines refer to Ott Meter No. 2311 and Price Meter No. 1553.
Dotted lines refer to an average for the two Ott Meters and the Price Meter.

TEST *V*.—AUGUST 12TH, 1914.

Horizontal.	Position and Meter.				
	1	2	3	4	5
	P	O_2	H_2	O_1	H_1
1	1 232	1 037	993	986	983
2	1 229	1 012	954	950	981
3	1 064	1 033	971	1 045	863
4	1 132	966	879	1 024	705
5	982	798	669	842	508
Totals	5 639	4 846	4 466	4 847	3 990
Means	2.820	2.423	2.233	2.424	1.995

turbulent water. It was thought that, to secure the most accurate results, one meter should be accelerated and the other retarded. However, as the Haskell and Ott meters were both used in each test, a diagram of their composite rating, Plate LVI, has been prepared. This diagram possesses the seeming inconsistency that the higher of two readings by the Haskell meter for a given reading by the Ott meter corresponds to the lower of the two velocities indicated by the diagram, and *vice versa*.

On each of the five diagrams of composite ratings the solid lines correspond to the composite ratings of only one pair of meters, and the broken lines correspond to the average composite ratings for the two types of meter represented, as determined from the Ann Arbor ratings.

Plates LII and LIV are for horizontal transverse oscillations only, and Plates LIII and LV are made from curves drawn midway between those for equal degrees of horizontal and vertical oscillations. The latter two diagrams, consequently, assume equal degrees of horizontal and vertical turbulence, but omit longitudinal turbulence, as has been explained in Section 122. The former pair of drawings assumes only horizontal turbulence, omitting longitudinal and vertical perturbations. Plate LVI is for the Haskell-Ott combination. As these meters are practically constant in form for all planes through their axes, they should be equally affected by equal

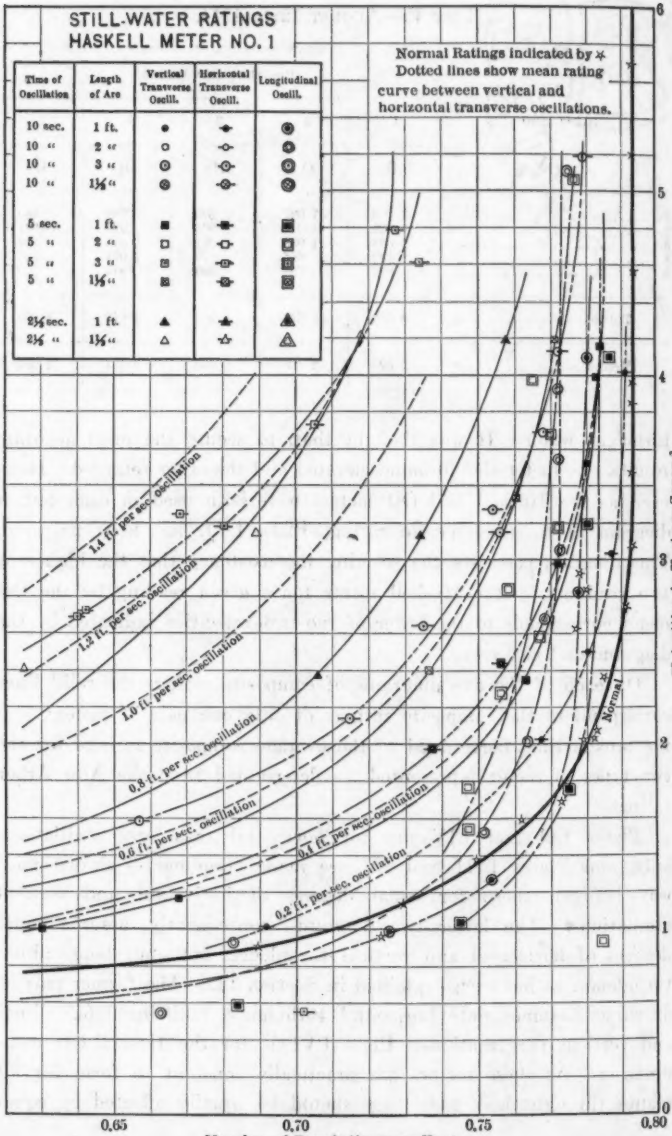

STILL-WATER RATINGS
HASKELL METER NO. 1
Time of Oscillation
Length of Arc
Vertical Transverse Oscill.
Horizontal Transverse Oscill.
Longitudinal Oscill.
10 sec. 1 ft.
10 " 2 "
10 " 3 "
10 " 1½ "
5 sec. 1 ft.
5 " 2 "
5 " 3 "
5 " 1½ "
2½ sec. 1 ft.
2½ " 1½ "
Normal Ratings indicated by ✳
Dotted lines show mean rating curve between vertical and horizontal transverse oscillations.
1.4 ft. per sec. oscillation
1.2 ft. per sec. oscillation
1.0 ft. per sec. oscillation
0.8 ft. per sec. oscillation
0.6 ft. per sec. oscillation
0.4 ft. per sec. oscillation
0.2 ft. per sec. oscillation
Normal
Velocities, in Feet per Second.
0
1
2
3
4
5
6
0.65
0.70
0.75
0.80
Number of Revolutions per Foot.

FIG. 32.

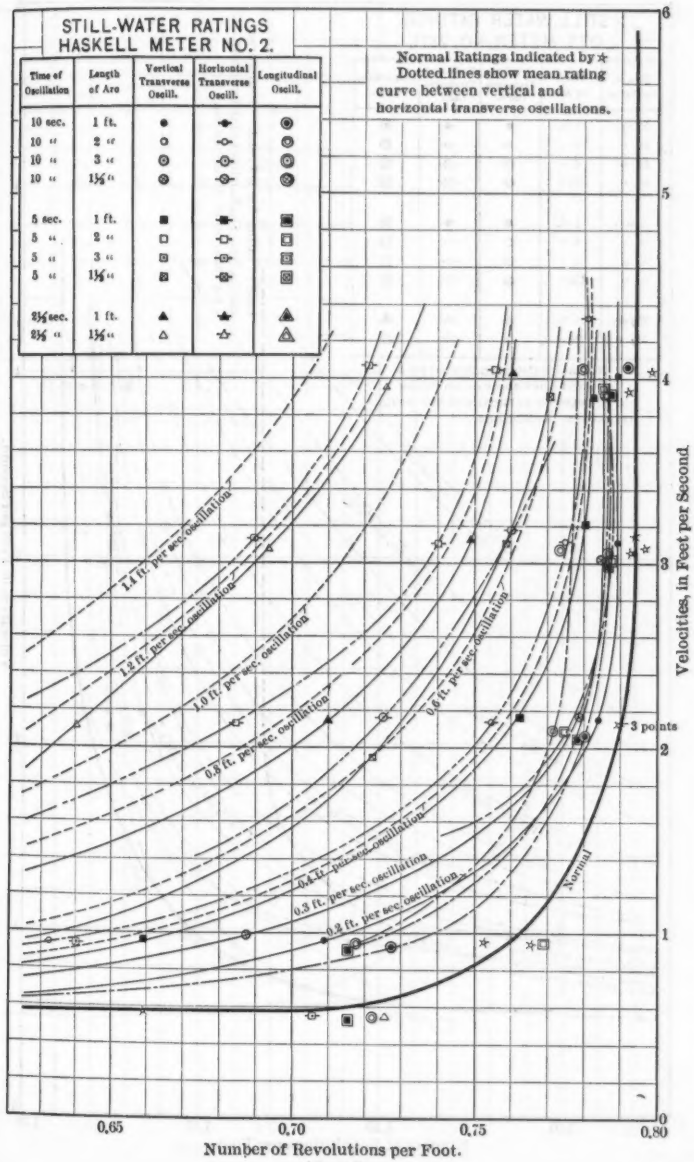

STILL-WATER RATINGS
HASKELL METER NO. 2.
Time of Oscillation
Length of Arc
Vertical Transverse Oscill.
Horizontal Transverse Oscill.
Longitudinal Oscill.
10 sec. 1 ft.
10 " 2 "
10 " 3 "
10 " 1½ "
5 sec. 1 ft.
5 " 2 "
5 " 3 "
5 " 1½ "
2½ sec. 1 ft.
2½ " 1½ "
Normal Ratings indicated by ☆
Dotted lines show mean rating curve between vertical and horizontal transverse oscillations.
1.4 ft. per sec. oscillation
1.2 ft. per sec. oscillation
1.0 ft. per sec. oscillation
0.8 ft. per sec. oscillation
0.6 ft. per sec. oscillation
0.4 ft. per sec. oscillation
0.3 ft. per sec. oscillation
0.2 ft. per sec. oscillation
Normal
3 points
Velocities, in Feet per Second
0
1
2
3
4
5
6
0.65
0.70
0.75
0.80
Number of Revolutions per Foot.

FIG. 33.

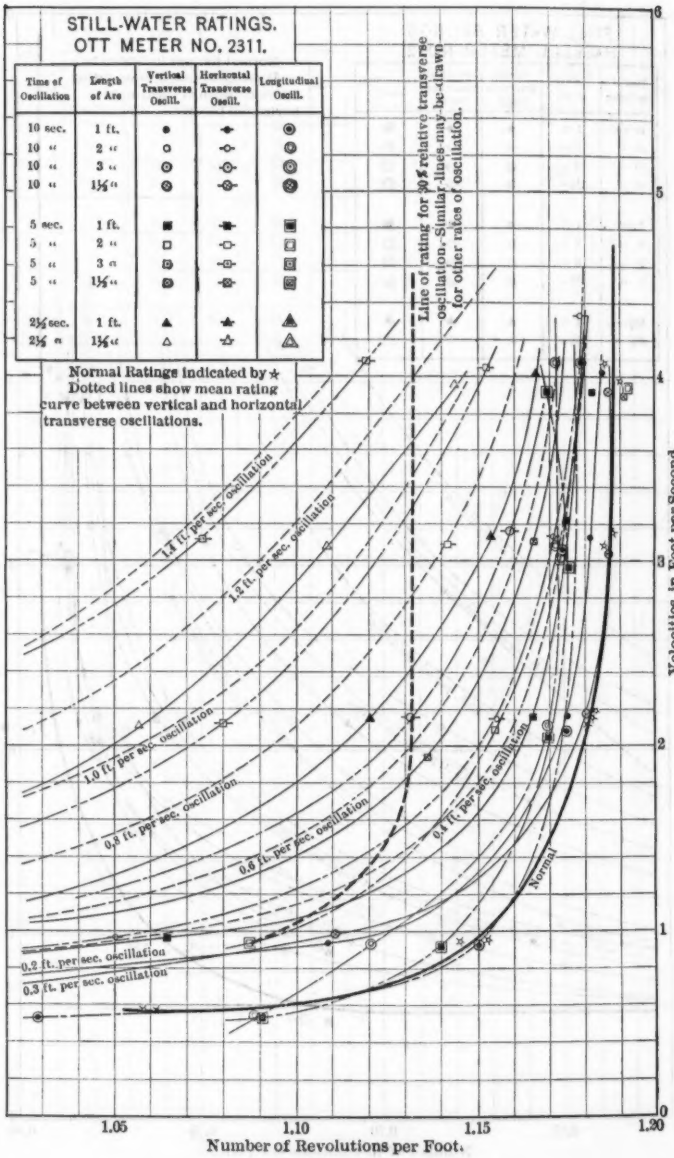

STILL-WATER RATINGS.
OTT METER NO. 2311.
Time of Oscillation
Length of Arc
Vertical Transverse Oscill.
Horizontal Transverse Oscill.
Longitudinal Oscill.
10 sec. 1 ft.
10 " 2 "
10 " 3 "
10 " 1½ "
5 sec. 1 ft.
5 " 2 "
5 " 3 "
5 " 1½ "
2½ sec. 1 ft.
2½ " 1½ "
Normal Ratings indicated by ☆
Dotted lines show mean rating curve between vertical and horizontal transverse oscillations.
Line of rating for 30° relative transverse oscillation. Similar lines may be drawn for other rates of oscillation.
1.4 ft. per sec. oscillation
1.2 ft. per sec. oscillation
1.0 ft. per sec. oscillation
0.8 ft. per sec. oscillation
0.6 ft. per sec. oscillation
0.4 ft. per sec. oscillation
Normal
0.2 ft. per sec. oscillation
0.3 ft. per sec. oscillation
Velocities, in Feet per Second.
0
1
2
3
4
5
6
1.05
1.10
1.15
1.20
Number of Revolutions per Foot.

FIG. 34.

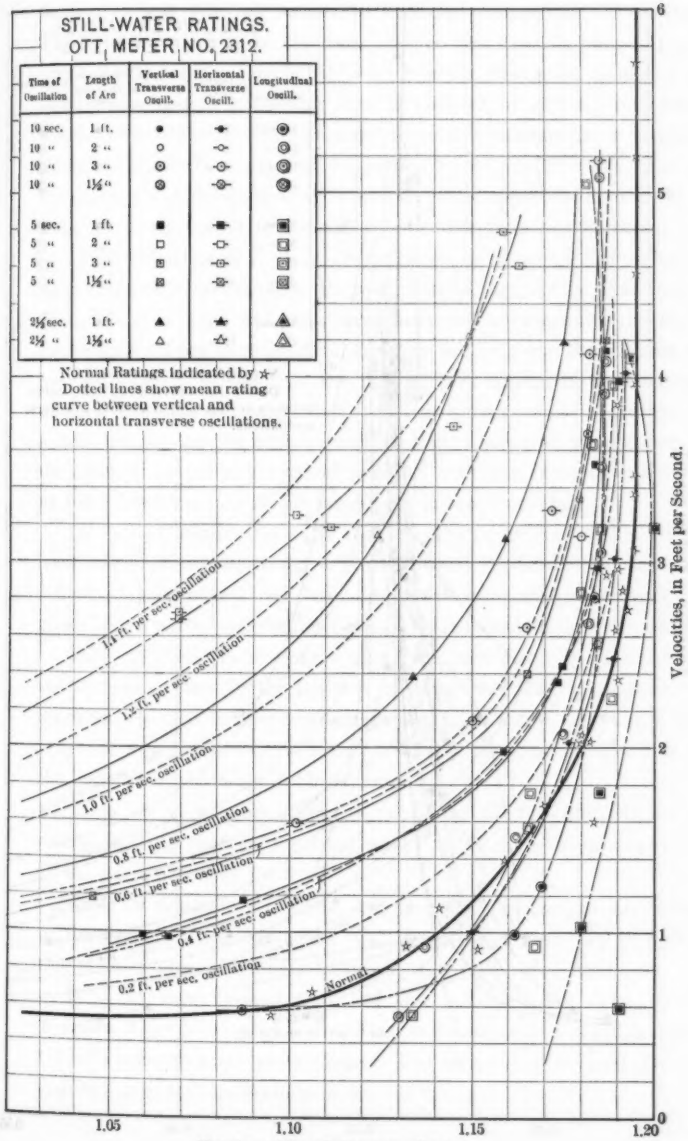

STILL-WATER RATINGS.
OTT METER NO. 2312.
Time of Oscillation
Length of Arc
Vertical Transverse Oscill.
Horizontal Transverse Oscill.
Longitudinal Oscill.
10 sec. 1 ft.
10 " 2 "
10 " 3 "
10 " 1½ "
5 sec. 1 ft.
5 " 2 "
5 " 3 "
5 " 1½ "
2½ sec. 1 ft.
2½ " 1½ "
Normal Ratings indicated by ☆
Dotted lines show mean rating curve between vertical and horizontal transverse oscillations.
1.4 ft. per sec. oscillation
1.2 ft. per sec. oscillation
1.0 ft. per sec. oscillation
0.8 ft. per sec. oscillation
0.6 ft. per sec. oscillation
0.4 ft. per sec. oscillation
0.2 ft. per sec. oscillation
Normal
Velocities, in Feet per Second.
0
1
2
3
4
5
6
1.05
1.10
1.15
1.20
Number of Revolutions per Foot.

FIG. 35.

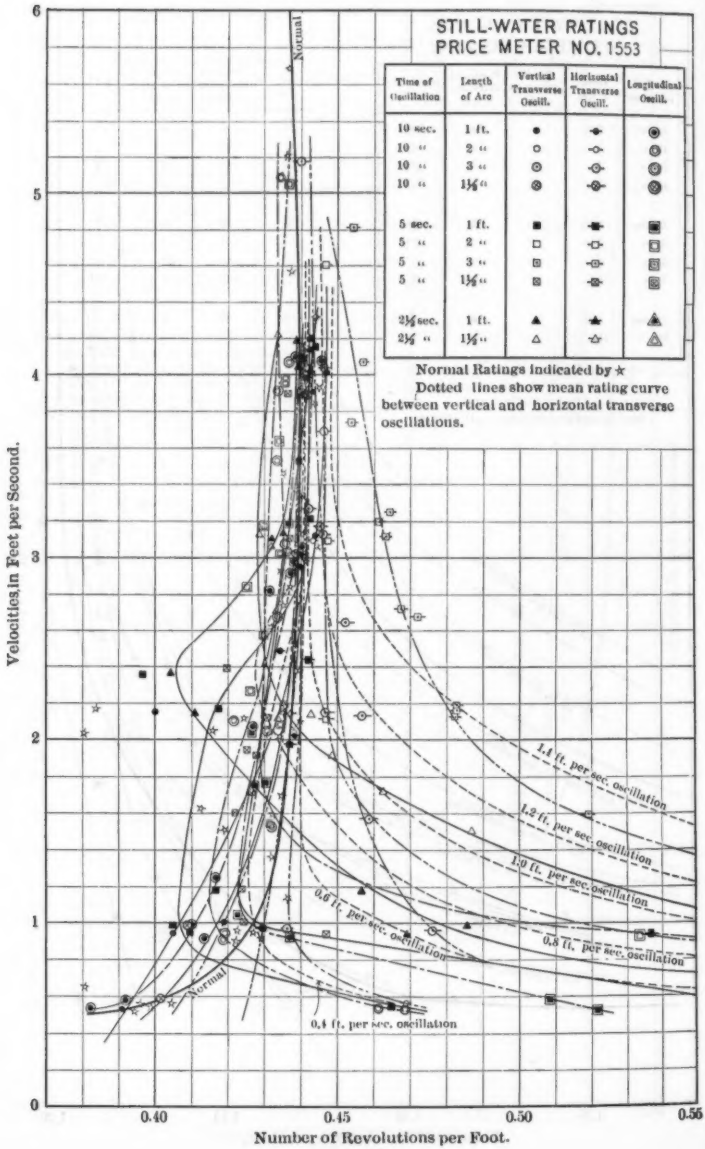

STILL-WATER RATINGS
PRICE METER NO. 1553
Time of Oscillation
Length of Arc
Vertical Transverse Oscill.
Horizontal Transverse Oscill.
Longitudinal Oscill.
10 sec. 1 ft.
10 " 2 "
10 " 3 "
10 " 1½ "
5 sec. 1 ft.
5 " 2 "
5 " 3 "
5 " 1½ "
2½ sec. 1 ft.
2½ " 1½ "
Normal Ratings indicated by ★
Dotted lines show mean rating curve between vertical and horizontal transverse oscillations.
Normal
Normal
1.4 ft. per sec. oscillation
1.2 ft. per sec. oscillation
1.0 ft. per sec. oscillation
0.8 ft. per sec. oscillation
0.6 ft. per sec. oscillation
0.4 ft. per sec. oscillation
Velocities in Feet per Second.
0
1
2
3
4
5
6
0.40
0.45
0.50
0.55
Number of Revolutions per Foot.

FIG. 36.

degrees of any kind of transverse turbulence, horizontal or vertical. The tests show this to be the case. Hence only one diagram is necessary for this combination of meters. It is the change in rating of the small Price meter in passing from horizontal to vertical turbulence which necessitates the two diagrams when this instrument is involved, though it must be remarked, in justice to this meter, that it is, on the average, for the method of suspension and operation used on these tests, relatively nearer to its still-water rating than any of the others.

124.—Velocities in Tests G, H, I, K, and L, Based on Meter Corrections Deduced from Average Meter Velocities in the Five Positions in These Tests.—These five tests were intended to determine the meter corrections. They were made under very uniform conditions, and are nearly identical, except that the meters were shifted in a systematic manner from test to test, so that each instrument made the traverse of a different position in each succeeding test, thereby operating once at the intersection of any vertical position with any horizontal, that is, at 100 different points of the head-race, in the five tests.

Instead of comparing each pair of meters at each point in the head-race, thus necessitating a large amount of detail, the aggregate velocity for each test has been computed from the mean velocities given by the partial aggregates for the separate meters in each position.

Table 67 gives the average reading for each meter in each test for the 20 meter points in the position in which the particular meter was operated, reduced to the common discharge of 1 660 cu. ft. per sec., and to the common section area of 625 sq. ft. See Section 121 for the data from which the table is compiled.

Table 68 is a rearrangement of Table 67 for the purpose of assembling the records for each meter in each position, regardless of the particular test involved.

Table 69 determines the true average velocities for the five meters in the five positions, regardless of the particular test. In compiling this table, Plates LII, LIII, LIV, LV, and LVI, were used.

It is at once evident that the two velocities for any position determined for horizontal and vertical oscillations agree within about 1% of each other, as an average. The same may be said for the case of horizontal oscillations only.

TABLE 67.—EQUIVALENT MEAN INSTRUMENTAL VELOCITIES IN TESTS *G, H, I, K,* AND *L,* FOR A DISCHARGE OF 1 660 CU. FT. PER SEC. AND AN AREA OF 625 SQ. FT., BASED ON STILL-WATER RATINGS. UNIT NO. 13.

Position number.	Test.									
	G		*H*		*I*		*K*		*L*	
1.....	O_1	2.73	O_2	2.71	H_2	2.65	P	2.83	H_1	2.71
2.....	H_1	2.48	O_1	2.47	O_2	2.41	H_2	2.44	P	2.63
3.....	P	2.53	H_1	2.33	O_1	2.36	O_2	2.37	H_2	2.34
4.....	H_2	2.38	P	2.83	H_1	2.45	O_1	2.53	O_2	2.47
5.....	O_2	2.52	H_2	2.34	P	2.88	H_1	2.18	O_1	2.57
Section area.....	625.8		625.8		625.2		623.9		626.0	
Approx. discharge.	1 662		1 662		1 666		1 653		1 647	
Factor*.....	1.001		1.001		0.997		1.003		1.010	

* This factor was used to reduce the meter record tests *G, H, I, K,* and *L,* Section 121, to values corresponding to a discharge of 1 660 cu. ft. per sec. and an area of 625 sq. ft. The corrections are so small that they are frequently omitted.

It may be said, therefore, that the two velocities determined by the Haskell-Price and Ott-Price combinations agree fairly well.

Velocities by the Haskell-Ott combination, however, are considerably deficient, as an inspection of the fifth line of velocities in Table 69 shows. This is undoubtedly due to some change in the relations between the two meters. The Ott meter appears to read lower than it should, according to the ratings, as all the velocities for the combinations which include the Ott meter are low.

It was stated in Section 123 that it was not intended to use velocities determined by the Haskell-Ott combination, because the two screw meters are so much alike in performance that errors become magnified. Hence the velocities by this combination are untrustworthy, and should be neglected, except for purposes of comparison.

The problem then narrows itself to the determination of correct average velocities by the separate combinations of the Haskell and Ott meters with the Price.

In the case of the diagrams for horizontal oscillations, it is clear that, if we should wish to alter them by introducing vertical components, the errors of the Price meter would be reduced and those

PLATE LIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXX, No. 1366.
GROAT ON
CHEMI-HYDROMETRY.

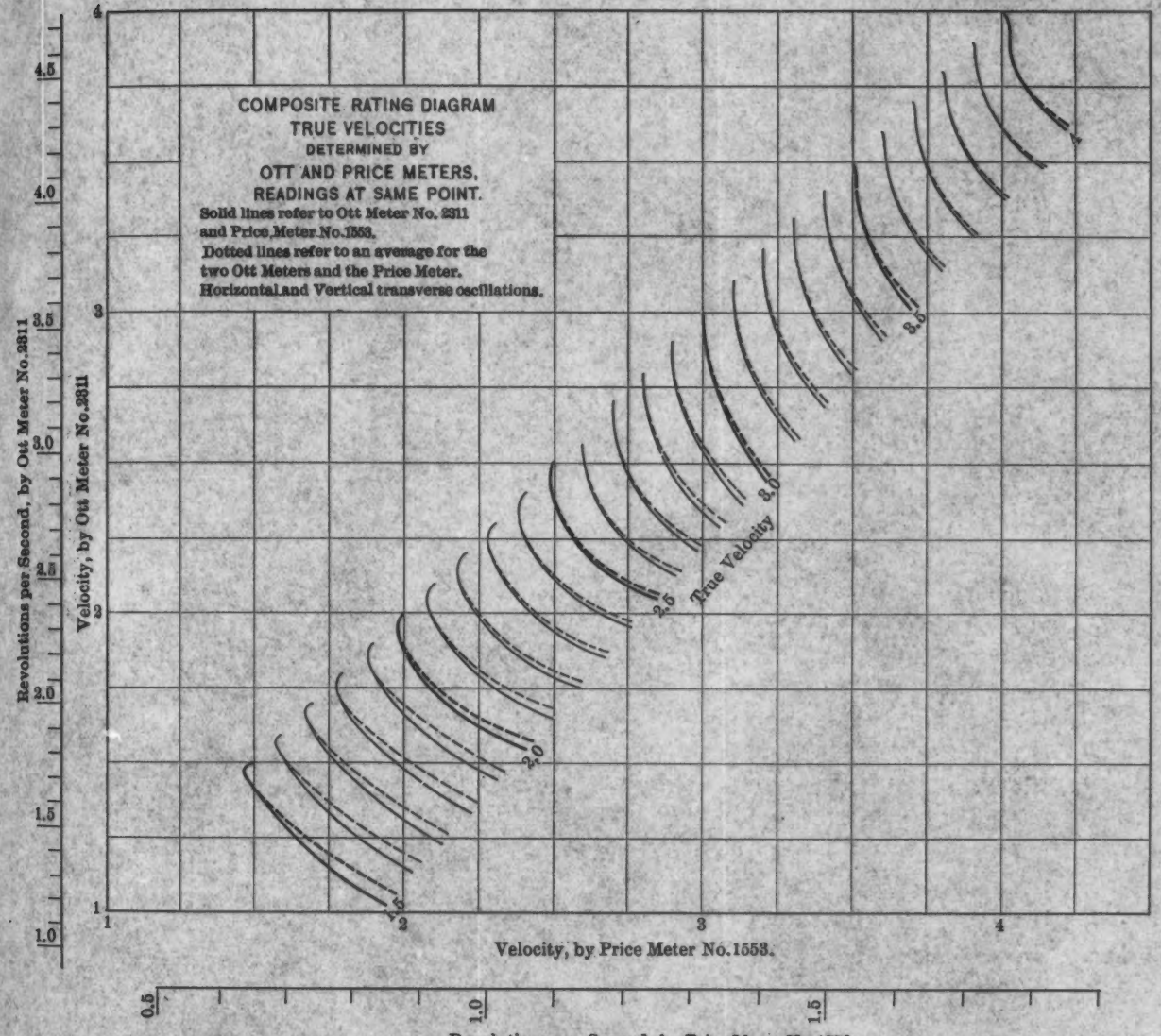


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TABLE 68.—MEAN INSTRUMENTAL VELOCITIES OF THE FIVE METERS IN THE FIVE POSITIONS FOR TESTS *G*, *H*, *I*, *K*, AND *L*.—A REARRANGEMENT OF TABLE 67.

(Common discharge = 1 660 cu. ft. per sec. Common area = 625 sq. ft.)

Meter.	Position.				
	1	2	3	4	5
Price	2.83	2.63	2.53	2.83	2.88
$Haskell_1$	2.71	2.48	2.33	2.45	2.18?
$Haskell_2$	2.65	2.44	2.34	2.38	2.34
Ott_1	2.73	2.47	2.36	2.53	2.57
Ott_2	2.71	2.41	2.37	2.47	2.52
Average Haskell	2.68	2.46	2.335	2.415	2.26?
Average Ott	2.72	2.44	2.365	2.50	2.545

TABLE 69.—TRUE AVERAGE VELOCITY DETERMINED BY PLATES LII, LIII, LIV, LV, AND LVI, FROM TABLE 68.

(Common discharge = 1 660 cu. ft. per sec. Common area = 625 sq. ft.)

Combination of meters.	Diagram.	Kind of oscillation.	Position.				
			1	2	3	4	5
H-P	Plate LV	Horizontal and vertical.	2.83	2.64	2.55	2.81	2.80 (2.84)
H-P	" LIV	Horizontal	2.80	2.61	2.50	2.73	2.70
O-P	" LIII	Horizontal and vertical	2.83	2.61	2.52	2.76	2.81
O-P	" LII	Horizontal	2.79	2.56	2.47	2.70	2.74 (2.75)
O H	" LVI		2.76	2.45?*	2.40	2.59	3.00
Average of all five			2.802	2.574	2.488	2.718	(2.776) 2.81
Average of upper four			2.812	2.605	2.51	2.75	(2.782) 2.762

*Meter readings are discordant. Ott should exceed Haskell. Mean of meter velocities taken, and this must be too low.

Haskell meter seems to run slow in Position 5, Test *K*. The figures in parentheses represent a possibly more correct value.

of the screw meters would be numerically increased. This would cause the true velocity curve scale to rotate about the true velocity points for zero errors (on the 45° line) in clockwise direction. Hence, if actual vertical components are present in the head-race, velocities

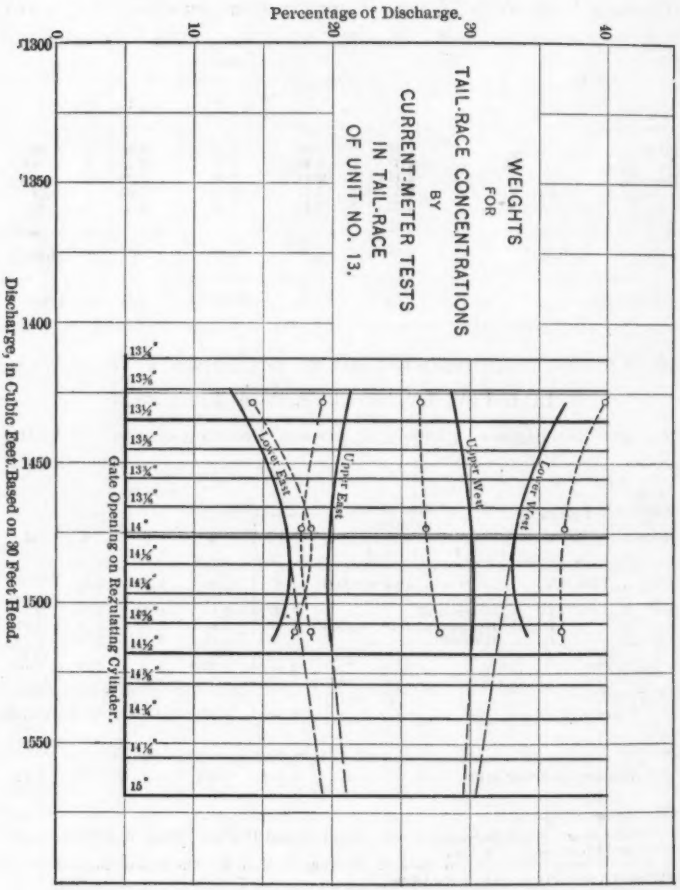

Percentage of Discharge.
0
10
20
30
40
1300
1350
1400
1450
1500
1550
Discharge, in Cubic Feet, Based on 30 Feet Head.
Gate Opening on Regulating Cylinder.
13¼"
13⅜"
13½"
13⅝"
13¾"
13⅞"
14"
14⅛"
14¼"
14⅜"
14½"
14⅝"
14¾"
14⅞"
15"
WEIGHTS FOR TAIL-RACE CONCENTRATIONS BY CURRENT-METER TESTS IN TAIL-RACE OF UNIT NO. 13.
Lower East
Upper East
Upper West
Lower West

FIG. 37.

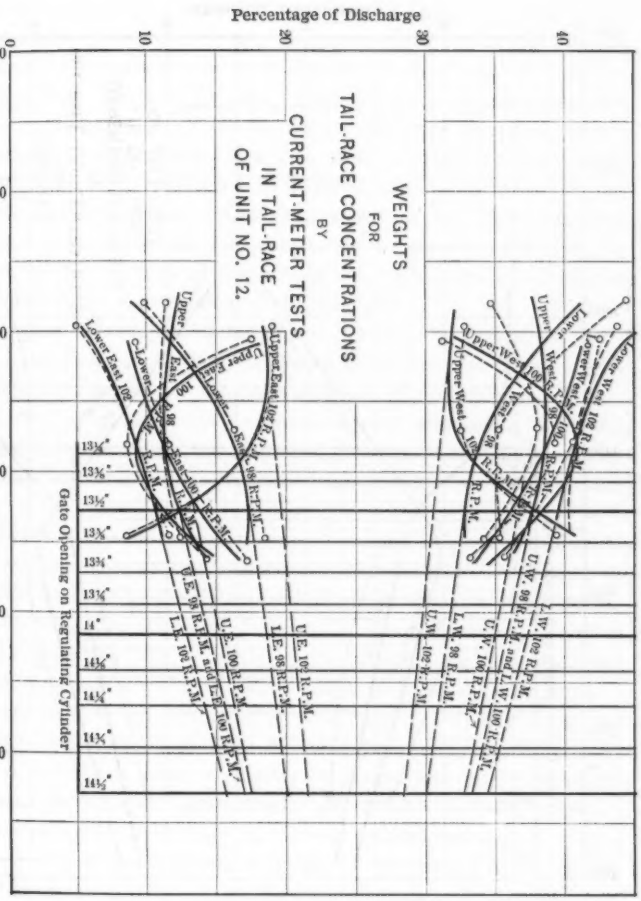

Percentage of Discharge
0
10
20
30
40
1300
1350
1400
1450
1500
1550
WEIGHTS
FOR
TAIL-RACE CONCENTRATIONS
BY
CURRENT-METER TESTS
IN TAIL-RACE
OF UNIT NO. 12.
Discharge, in Cubic Feet, Based on 30 Feet Head.
Gate Opening on Regulating Cylinder
13¼"
13⅜"
13½"
13⅝"
13¾"
13⅞"
14"
14⅛"
14¼"
14⅜"
14½"

FIG. 38.

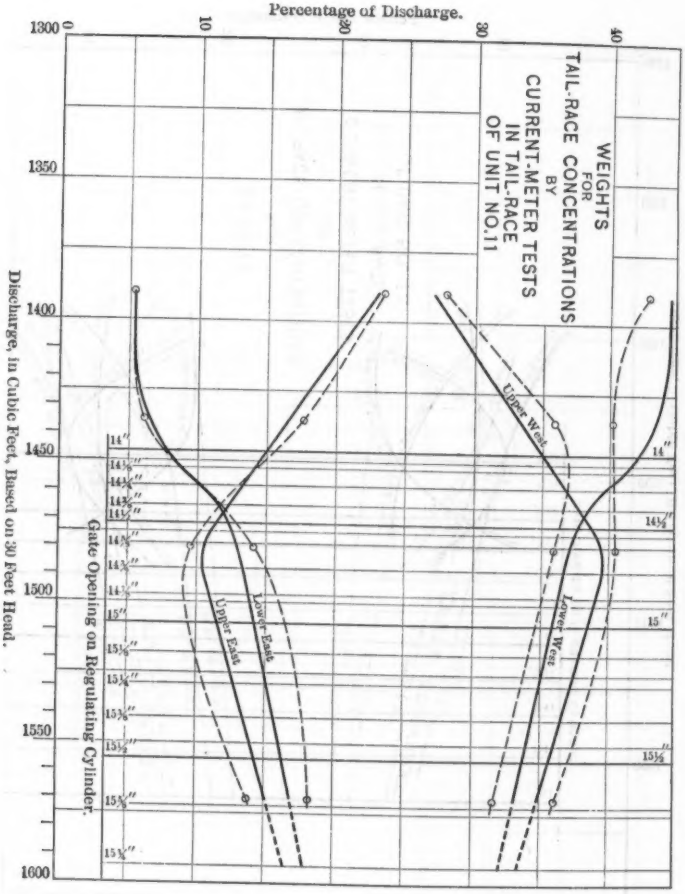

WEIGHTS FOR TAIL-RACE CONCENTRATIONS BY CURRENT-METER TESTS IN TAIL-RACE OF UNIT NO. 11
Percentage of Discharge.
0
10
20
30
40
1300
1350
1400
1450
1500
1550
1600
Discharge, in Cubic Feet, Based on 30 Feet Head.
Gate Opening on Regulating Cylinder.
14"
14½"
14⅝"
14¾"
14⅞"
15"
15⅛"
15¼"
15⅜"
15½"
15⅝"
15¾"
Upper West
Upper East
Lower East
Lower West

FIG. 39.

determined by the diagrams for horizontal oscillations will be too low. There are undoubtedly vertical components of motion in the headwater. Therefore, all the velocities by the diagrams for horizontal disturbance should be too low, rather than too high.

By similar reasoning, it may be shown that, in the case of the diagrams for horizontal and vertical components, if too little oscillation has been assumed for the screw meters in constructing the diagrams, the curve scales will lie counter clockwise from their proper positions. In constructing these diagrams, only the horizontal components have been considered in plotting the co-ordinates for the Haskell and Ott meters. Therefore, velocities by these diagrams, also, will tend to be too low, supposing the correct values of the components have been used in plotting the co-ordinates for the Price meter during the construction of the diagrams.

Moreover, all readings of the meters are likely to be low, as longitudinal oscillations have been neglected altogether. It is not likely, therefore, that any of the velocities in Table 69 are too high.

On the other hand, if too much vertical component has been assumed in constructing the diagrams, the lines would lie clockwise from their correct positions, thus resulting in velocities which are too high.

Now, there was probably not as much vertical as horizontal disturbance in the water, so one element is present which would tend to make the readings by the diagrams for horizontal and vertical oscillations too high. On the whole, it might be assumed, without a serious probable error, that the velocities by the diagrams for horizontal and vertical components will give results which are nearest to the truth.

There will be no serious objection, however, to averaging the velocities in the first four lines of Table 69, as no great differences are involved. In this way a possible systematic error of some magnitude may be reduced. Accordingly, an average of those velocities for each position will be taken as the best probable value.

We are now in a position to compute the factors by which the velocities of the various meters for each position must be multiplied to secure true velocities. The true average velocities at the foot of Table 69 are to be divided by the average velocities of each type of meter from Table 68 to determine the factors. The result is given in Table 70.

TABLE 70.—CORRECTION FACTORS FOR THE METERS, BASED ON THE COMPUTED TRUE VELOCITIES FOR THE FIVE METER POSITIONS IN TESTS *G*, *H*, *I*, *K*, AND *L*, DETERMINED BY PLATES LII, LIII, LIV, AND LV.

(Common discharge = 1 600 cu. ft. per sec. Common area = 625 sq. ft.)

Meter.	Position of Meter.					
	1	2	3	4	5	Mean.
Price	0.994	0.991	0.992	0.972	(0.967) 0.960 (1.179)	0.982
Haskell	1.049	1.059	1.075	1.139	1.222 (1.093)	1.109
Ott	1.034	1.068	1.061	1.100	1.085	1.070
Mean						1.054

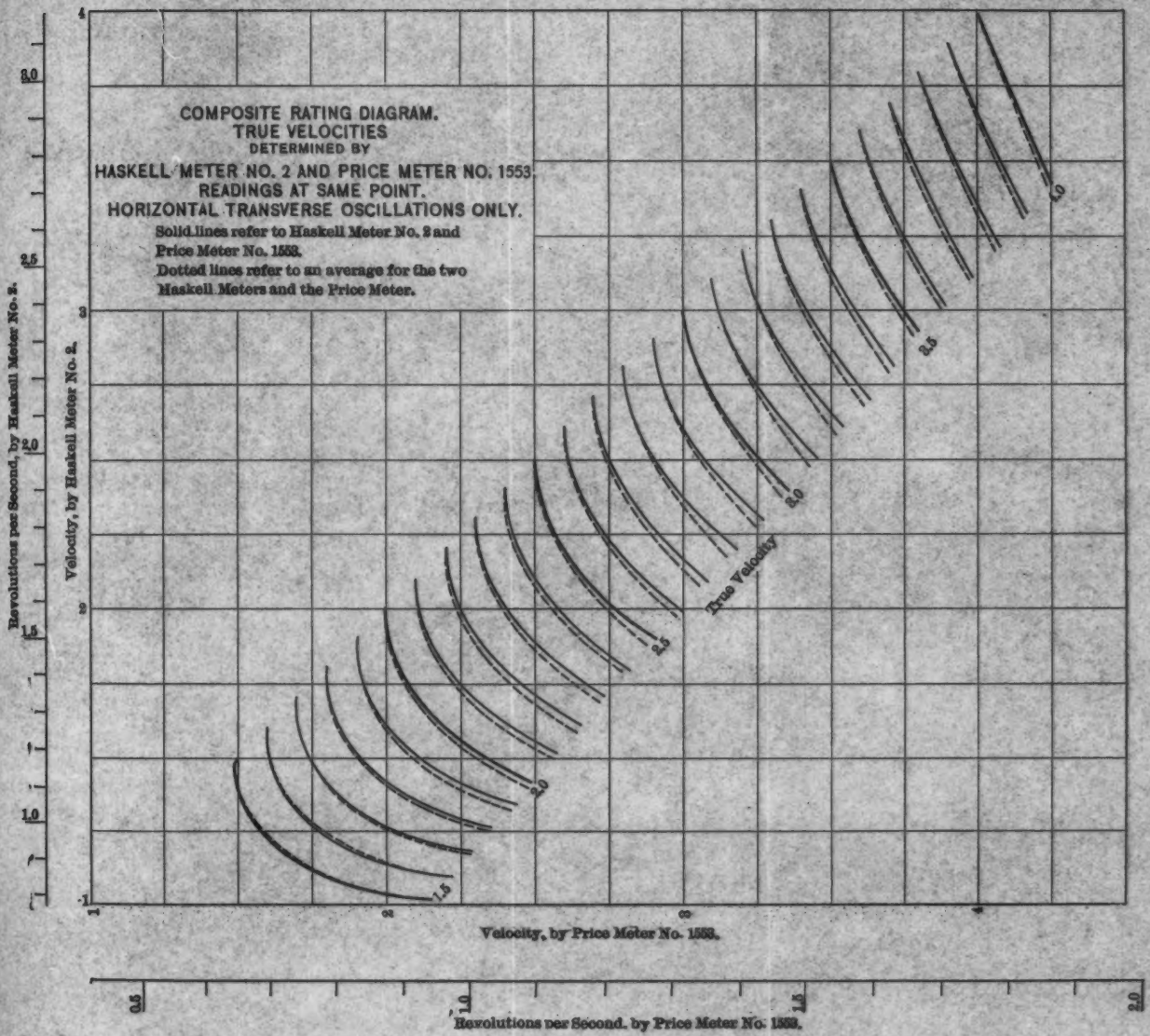
NOTE.—The parentheses refer to a possibly more correct value. See foot-note to Table 69.

TABLE 71.—SUMS OF THE TRUE VELOCITIES AT THE 100 METER POINTS IN TESTS *G*, *H*, *I*, *K*, AND *L*, BASED ON FACTORS DETERMINED FROM THE PERFORMANCE OF THE METERS IN THE FIVE POSITIONS DURING THESE TESTS.

Position.	Test.				
	G	*H*	*I*	*K*	*L*
1	505	559	557	502	562
2	524	528	515	516	516
3	503	501	503	501	499
4	541	549	559	555	538
5	(550) 546	(552) 572	(558) 553	(513) 532	(566) 552
Sum of velocities at the 100 meter points in head-race.	(268.3) 267.9	(268.9) 270.9	(269.1) 268.6	(264.7) 266.6	(267.1) 266.7

NOTE.—The figures in parentheses are based on a possibly more correct value for meter velocity. See foot-note to Table 69.

Turning now to the tests in Section 121, it is a simple matter to compute all the sums of the true velocities for the five positions of each test by multiplying the five totals by the corresponding factors from Table 70. This process leads to Table 71, the totals therein being the computed sums of the velocities at the 100 meter points,


COMPOSITE RATING DIAGRAM.
TRUE VELOCITIES
DETERMINED BY
HASKELL METER NO. 2 AND PRICE METER NO. 1553.
READINGS AT SAME POINT.
HORIZONTAL TRANSVERSE OSCILLATIONS ONLY.
Solid lines refer to Haskell Meter No. 2 and Price Meter No. 1553.
Dotted lines refer to an average for the two Haskell Meters and the Price Meter.
Revolutions per Second, by Haskell Meter No. 2.
Velocity, by Haskell Meter No. 2.
Velocity, by Price Meter No. 1553.
Revolutions per Second, by Price Meter No. 1553.
True Velocity
1.5
2.0
2.5
3.0
3.5
4.0
0.5
1.0
1.5
2.0

based on the factors determined in Table 70 from a comparison of velocities by the different meters in each position of Tests *G*, *H*, *I*, *K*, and *L*.

125.—Velocities in Tests G, H, I, K, and L, Based on Meter Corrections Deduced from Average Meter Velocities in the Five Horizontals in These Tests.—To show that we have a fairly stable method of computing factors for connecting the meter velocities, it will be advisable to compute them for the five horizontals, using the data from the same tests treated in the preceding section. It will not be necessary, however, in the case of horizontals, to reduce to a common discharge and common area, as each meter operates once in each position during the five tests. Variations due to differences in conditions among the tests tend to affect all meters in a similar sense, thus eliminating, on the average, any serious inconsistencies due to this cause. The average velocity for each meter in a given horizontal for all five tests is computed and tabulated, thus leading to a table similar to Table 68. For example, the average velocity in Tests *G*, *H*, *I*, *K*, and *L* for Ott meter No. 1, in Horizontal 1, would result from the sums of the velocities taken from the tests, Section 121, as follows: $(10.98 + 10.32 + 10.00 + 10.94 + 10.30) \div 20 = 2.627$. This figure, rounded off to 2.63, may then be entered in its proper place in Table 72.

TABLE 72.—MEAN INSTRUMENTAL VELOCITIES BY THE FIVE METERS IN THE FIVE HORIZONTALS, FOR TESTS *G*, *H*, *I*, *K*, AND *L*.

Meters.	Horizontal.				
	1	2	3	4	5
P.	2.88	2.91	2.92	2.57	2.39
H_1	2.60	2.64	2.76	2.34	1.77
H_2	2.59	2.78	2.65	2.27	1.84
O_1	2.63	2.71	2.73	2.60	1.97
O_2	2.64	2.67	2.57	2.59	1.96
Average H	2.60	2.71	2.70	2.31	1.80
Average O	2.63	2.69	2.65	2.59	1.97

Just as Table 69 was compiled from Table 68 we may arrive at Table 73 for true average velocities in the horizontals during the five tests.

TABLE 73.—TRUE AVERAGE VELOCITIES DETERMINED BY PLATES LII, LIII, LIV, LV, AND LVI, FROM TABLE 72.

Combination of meters.	Diagram.	Kind of oscillation.	HORIZONTAL.				
			1	2	3	4	5
H-P	Plate LV	Hor. and Vert.	2.87	2.90	2.91	2.57	2.28
H-P	" LIV	Horizontal	2.82	2.88	2.88	2.52	2.22
O-P	" LIII	Hor. and Vert.	2.84	2.88	2.87	2.58*	2.25
O-P	" LII	Horizontal	2.78	2.83	2.82	2.58†	2.20
O-H	" LVI		2.67	2.70*	2.68*	(3.05) 3.10†	2.20
Mean of first four			2.828	2.872	2.870	2.562	2.238

* Price reads lower than Ott, or Ott reads lower than Haskell. Mean of meter velocities taken.

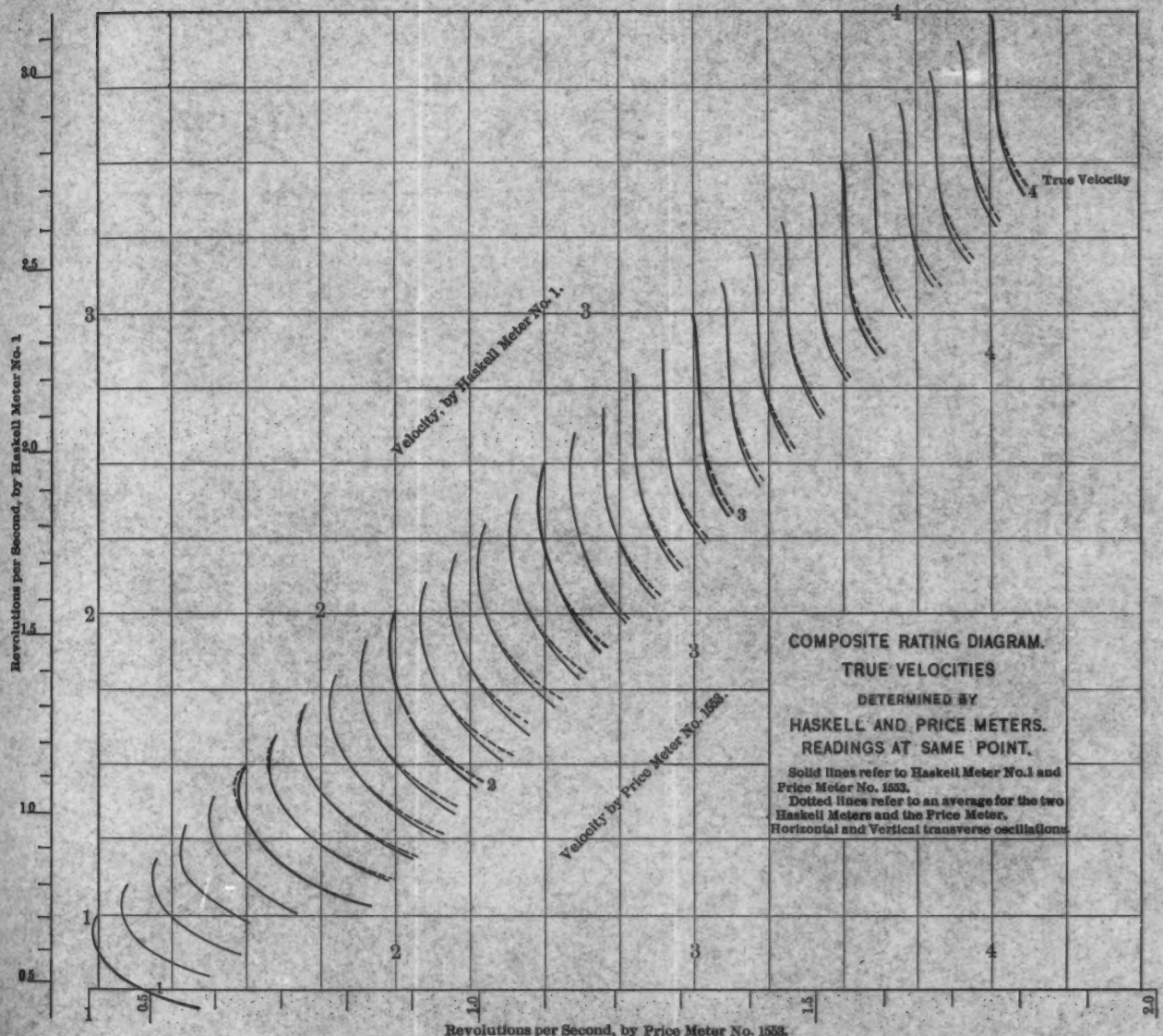
Continuing, as in Section 124, the factors of Table 74 have been computed.

TABLE 74.—CORRECTION FACTORS FOR THE METERS, BASED ON THE COMPUTED TRUE VELOCITIES FOR THE FIVE METER HORIZONTALS IN TESTS *G*, *H*, *I*, *K*, AND *L*, DETERMINED BY PLATES LII, LIII, LIV, LV, AND LVI.

Meter.	HORIZONTAL.					
	1	2	3	4	5	Mean.
Price	0.981	0.988	0.985	0.996	0.936	0.977
Haskell	1.089	1.060	1.062	1.112	1.240	1.113
Ott	1.074	1.068	1.082	0.988*	1.138	1.070
Mean						1.053

* This, of course, is abnormal. A further study would be required to eradicate the error.

Table 74 may then be used to calculate the sums of the velocities for each of the tests, noting that there is only one position in each test for a given meter in a particular horizontal, thus necessitating 25 multiplications to compute each test. We might have compared the meters for the intersection of each horizontal with each vertical position, thus obtaining 25 factors for the 25 meter rectangles in which each meter was operated, but this would have added considerably to the labor of the computation. Undoubtedly, a correction factor


Revolutions per Second, by Haskell Meter No. 1
Revolutions per Second, by Price Meter No. 1553.
Velocity by Haskell Meter No. 1.
Velocity by Price Meter No. 1553.
True Velocity
COMPOSITE RATING DIAGRAM.
TRUE VELOCITIES
DETERMINED BY
HASKELL AND PRICE METERS.
READINGS AT SAME POINT.
Solid lines refer to Haskell Meter No.1 and Price Meter No. 1553.
Dotted lines refer to an average for the two Haskell Meters and the Price Meter.
Horizontal and Vertical transverse oscillations.

for each of the 100 meter points in the head-race can be determined for each meter.

Table 75 illustrates the method in the computation of the sum of the velocities at the 100 meter points of Test *G*.

TABLE 75.—True Velocities for the 25 Intersections of Verticals and Horizontals in Test *G*, Based on the Factors of Table 74.

(Velocities, in feet per second.)

Horizontal.	Position and Meter.					
	1 — O_1	2 — H_1	3 — P	4 — H_2	5 — O_2	Total.
1	1 179	1 125	1 018	1 105	1 147	.....
2	1 311	1 115	1 045	1 017	1 090	.....
3	1 340	1 094	1 017	1 047	1 118	.....
4	1 058	1 125	988	1 092	1 172	.....
5	943	1 022	886	1 003	820	.....
	5 831	5 481	4 954	5 264	5 347	268.77

The aggregates of velocity by this method are:
G = 268.8 ft. per sec.
H = 268.3 " " "
I = 270.4 " " "
K = 261.0 " " "
L = 267.8 " " "

126.—Computations for Tests G, H, I, K, and L, with Factors Based on Tests N, O, P, Q, and R.—The computations in Tables 76 to 83 have been made in a manner similar to those of the preceding two sections.

TABLE 76.—Mean Instrumental Velocities by the Five Meters in the Five Positions for Tests *N*, *O*, *P*, *Q*, and *R*.

(Compare Table 68. Common discharge = 1 780 cu. ft. per sec. Common area = 619 sq. ft. Velocities, in feet per second.)

Meter.	Position.				
	1	2	3	4	5
Price	3.16	3.05	2.88	2.90	3.05
Haskell$_1$	2.96	2.65	2.63	2.63	2.62
Haskell$_2$	2.92	2.58	2.57	2.48	2.55
Ott$_1$	2.95	2.79	2.67	2.71	2.65
Ott$_2$	2.89	2.75	2.68	2.64	2.62
Average Haskell	2.94	2.62	2.60	2.56	2.58
Average Ott	2.92	2.77	2.68	2.68	2.64

TABLE 77.—TRUE AVERAGE VELOCITIES DETERMINED BY PLATES LII, LIII, LIV, AND LV, FROM TABLE 76.

(Common discharge = 1 780 cu. ft. per sec. Common area = 619 sq. ft. Velocities, in feet per second.)

Combinations of meters.	Diagram.	Oscillations.	Position.				
			1	2	3	4	5
H-P	Plate LV	Hor. and Vert.	3.14	3.02	2.87	2.89	3.02
H-P	" LIV	Horizontal	3.12	2.94	2.82	2.82	2.93
O-P	" LIII	Hor. and Vert.	3.13	3.01	2.86	2.87	2.97
O-P	" LII	Horizontal	3.06	2.94	2.81	2.82	2.88
Means			3.11	2.98	2.84	2.85	2.95

TABLE 78.—CORRECTION FACTORS FOR THE METERS BASED ON THE COMPUTED TRUE VELOCITIES FOR THE FIVE METER POSITIONS IN TESTS *N*, *O*, *P*, *Q*, AND *R*, DETERMINED BY PLATES LII, LIII, LIV, AND LV.

(Common discharge = 1 780 cu. ft. per sec. Common area = 619 sq. ft.)

Meter.	Position.					
	1	2	3	4	5	Mean.
Price	0.984	0.977	0.987	0.983	0.968	0.980
Haskell	1.058	1.138	1.092	1.113	1.143	1.109
Ott	1.065	1.076	1.060	1.063	1.118	1.076
Mean						1.055

TABLE 79.—AGGREGATES OF VELOCITY FOR TESTS *G*, *H*, *I*, *K*, AND *L* BY FACTORS BASED ON TESTS *N*, *O*, *P*, *Q*, AND *R*. METERS COMPARED BY POSITIONS.

Test.	Sum of velocities, in feet per second.
G	273.6
H	270.6
I	268.7
K	264.4
L	267.1

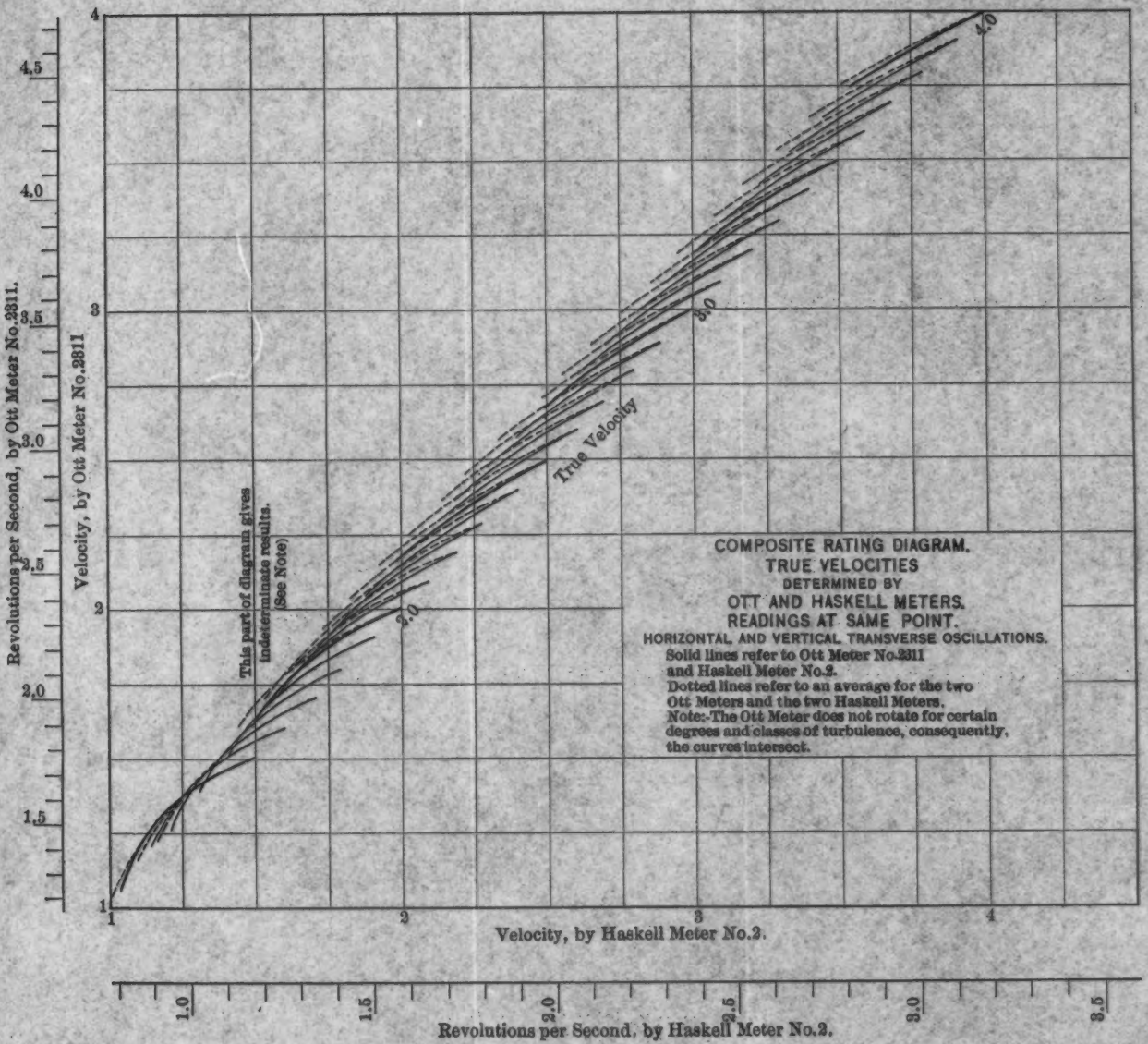

Revolutions per Second, by Ott Meter No.2311.
Velocity, by Ott Meter No.2311
Velocity, by Haskell Meter No.2.
Revolutions per Second, by Haskell Meter No.2.
True Velocity
This part of diagram gives indeterminate results. (See Note)
4.0
3.0
2.0
COMPOSITE RATING DIAGRAM.
TRUE VELOCITIES
DETERMINED BY
OTT AND HASKELL METERS.
READINGS AT SAME POINT.
HORIZONTAL AND VERTICAL TRANSVERSE OSCILLATIONS.
Solid lines refer to Ott Meter No.2311 and Haskell Meter No.2.
Dotted lines refer to an average for the two Ott Meters and the two Haskell Meters.
Note:-The Ott Meter does not rotate for certain degrees and classes of turbulence, consequently, the curves intersect.

TABLE 80.—MEAN INSTRUMENTAL VELOCITIES BY THE FIVE METERS IN THE FIVE HORIZONTALS FOR TESTS *N*, *O*, *P*, *Q*, AND *R*.

(Velocities, in feet per second.)

Meter.	Horizontal.				
	1	2	3	4	5
Price	3.16	3.08	3.18	2.94	2.66
Haskell$_1$	2.88	2.88	2.88	2.73	2.14
Haskell$_2$	2.82	2.80	2.75	2.64	2.11
Ott$_1$	2.94	2.87	2.89	2.79	2.29
Ott$_2$	2.93	2.82	2.91	2.74	2.22
Average Haskell	2.85	2.84	2.82	2.68	2.12
Average Ott	2.94	2.84	2.90	2.76	2.26

TABLE 81.—TRUE AVERAGE VELOCITIES DETERMINED BY PLATES LII, LIII, LIV, LV, AND LVI, FROM TABLE 80.

(Velocities, in feet per second.)

Combination of meters.	Diagram.	Kind of oscillation.	Horizontal.				
			1	2	3	4	5
H-P	Plate LV	Hor. and Vert.	3.14	3.07	3.16	2.93	2.59
H-P	" LIV	Horizontal.	3.09	3.03	3.09	2.89	2.51
O-P	" LIII	Hor. and Vert.	3.13	3.05	3.14	2.93	2.56
O-P	" LII	Horizontal.	3.07	2.99	3.07	2.87	2.49
O-H	" LVI		3.02	2.84	2.97	2.83	2.40
Average of first four			3.108	3.035	3.115	2.905	2.538

TABLE 82.—CORRECTION FACTORS FOR THE METERS, BASED ON THE COMPUTED TRUE VELOCITIES FOR THE FIVE METER HORIZONTALS IN TESTS *N*, *O*, *P*, *Q*, AND *R*, DETERMINED BY PLATES LII, LIII, LIV, LV, AND LVI.

Meter.	Horizontal.					
	1	2	3	4	5	Mean.
Price	0.984	0.985	0.980	0.988	0.954	0.978
Haskell	1.090	1.068	1.105	1.084	1.197	1.109
Ott	1.057	1.068	1.074	1.053	1.123	1.075
Mean						1.054

TABLE 83.—AGGREGATES OF VELOCITY FOR TESTS *G*, *H*, *I*, *K*, AND *L*, BY FACTORS BASED ON TESTS *N*, *O*, *P*, *Q*, AND *R*. METERS COMPARED BY HORIZONTALS.

Test.	Sum of velocities, in feet per second.
G	269.3
H	269.2
I	271.8
K	261.6
L	268.3

127.—Discharges in Tests G, H, I, K, and L.—Four different velocity computations have now been made for Tests *G*, *H*, *I*, *K*, and *L*. It will be of value to compare these velocities before computing the discharges. Some idea can be gained thereby of the degree of precision attained by the current-meter observations and calculations. The comparison is made in Table 84.

TABLE 84.—AGGREGATE VELOCITIES AT THE 100 UNIFORMLY DISTRIBUTED POINTS IN THE HEAD-RACE OF UNIT NO. 13 FOR TESTS *G*, *H*, *I*, *K*, AND *L*. THESE TESTS WERE NEARLY IDENTICAL AS TO CONDITIONS OF OPERATION.

(Velocities, in feet per second.)

Nature of factors for correcting the meter records.	TEST				
	G	*H*	*I*	*K*	*L*
Based on average velocities in the five positions for Tests *G*, *H*, *I*, *K*, and *L*, reduced to common area and discharge	267.9	270.9	268.6	266.6	266.7
Based on average velocities by each meter in each horizontal for the five tests, *G*, *H*, *I*, *K*, and *L*. Not reduced to common area and discharge	268.8	268.3	270.4	261.0	267.8
Based on average velocities in the five positions for Tests *N*, *O*, *P*, *Q*, and *R*, roughly reduced to common area and discharge	273.6	270.6	268.7	264.4	267.1
Based on average velocities by each meter in each horizontal for the five tests, *N*, *O*, *P*, *Q*, and *R*. Not reduced to common area and discharge	269.3	269.2	271.8	261.6	268.3
Averages	269.9	269.8	269.9	263.4	267.5

By referring to Tables 70, 74, 78, and 82, it will be seen that the average values of the correction factors for all meters are, respectively, 1.054, 1.053, 1.055, and 1.054, among which the maximum variation is found to be not more than 0.2 per cent. As to an average value, the system of factors is very stable.

There are relatively larger variations, however, among the four estimates of the sum of velocities for any individual test. As a general rule, the sum of the velocities for any test is larger when based on factors determined from Tests *N*, *O*, *P*, *Q*, and *R* than when based on those determined from Tests *G*, *H*, *I*, *K*, and *L*, though, on the average, this discrepancy is not large, the total sum for the five tests for the four classes of factors differing only as in Table 85.

TABLE 85.—TOTAL SUM OF VELOCITIES FOR TESTS *G*, *H*, *I*, *K*, AND *L*, BY THE FOUR METHODS.

(Velocities, in feet per second.)

Factors based on tests:	Meters compared by:	Total sum of velocities, all five tests.
G, *H*, *I*, *K*, *L*	Positions	1 340.7
G, *H*, *I*, *K*, *L*	Horizontals	1 336.3
N, *O*, *P*, *Q*, *R*	Positions	1 344.4
N, *O*, *P*, *Q*, *R*	Horizontals	1 340.2

As to individual tests, it will be seen that there are maximum variations for *G*, *H*, *I*, *K*, and *L*, respectively, as follows: 2.1, 1.0, 1.2, 2.1, and 0.6 per cent. Thus, different computations of the sum of the velocities, and therefore of the discharge, may vary by 1 or 2 per cent. The best that can be done by the foregoing computations will be to take the average sum of the velocities for each test at the bottom of Table 84 for the calculation of the corresponding discharge.

Before making the calculation, it must be explained that there are two corrections to be applied to current-meter discharges before the best estimate can result.

128.—Pump Discharge Added to Current-Meter Discharge.—In Section 51 it was shown that the discharge of the centrifugal pump was about 900 gal. per min., or about 2 cu. ft. per sec. As the suction of the pump was up stream from the current meters, and the discharge was down stream, this 2 cu. ft. per sec. must be added to the discharge determined by the current meters in tests where the chemical and meter methods were run in parallel.

129.—Meter-Rack Correction.—It was suspected by the writer that the frame for supporting the meters had a retarding reaction on the speed of the meters, as the framework of the rack formed a considerable obstruction in the head-race.

Three comparative tests were run to determine the amount of retardation, but none was executed under satisfactory conditions, and reliable information cannot now be given concerning the effect in question. However, the tests will be described briefly.

The tendency of the meter rack is to cut off the flow through the obstructed portion of the race and divert it to the unobstructed portion. Therefore, a meter in front of the rack will tend to run slower, and one in the section above or below the rack will tend to run faster, than with the rack removed.

Accordingly, some arms were made for the purpose of attaching the meters to the rack in such a manner as to hold them exactly 2 ft. vertically under their usual positions in front of the rack. A traverse of the race with the meters attached to the arms was then made, after which a traverse was made with the meters in their usual positions. The results of the traverses show that the meters were retarded, but the tests were too limited to give an exact measure. One of the tests is probably excessive and one is deficient in velocity, the mean of the three tests probably being not far from the truth, though it would seem to the writer to be too small rather than too large.

TABLE 86.—Comparison of Velocities at Points in the Head-Race, When the Meter Supporting the Rack Is in the Usual Position, with Those at the Same Points When the Rack is 2 ft. Higher, the Meters Being Then Held on Arms Extending 2 ft. Below the Rack.

Meters on Arms.		Meters in Normal Position.	
Test.	Mean velocity, in feet per second.	Test.	Mean velocity, in feet per second.
24	2.494	25	2.450
71	2.559	70	2.601
72	2.690	73	2.668
Mean	2.581	Mean	2.573

On the average, there is an apparent retardation of 0.3%, due to the presence of the rack.

It may be explained that the meters always operated at the regular meter points in the race, the effect being that the rack was 2 ft. higher than its usual position when the meters were attached to the arms.

The method of suspending and operating the meters is explained fully in Part III.

130.—Computation of Discharge in Tests G, H, I, K, and L.—To obtain the metered discharges for these tests from the average sums of the velocities in Table 84, it is merely necessary to multiply the sums by the distribution factor and the resulting product by the area of the section. By applying additively about 7 cu. ft. per sec., the corrected discharge may be obtained, taking into account the pump and rack corrections. Table 87 results.

TABLE 87.—DISCHARGE IN TESTS *G, H, I, K,* AND *L,* BASED ON AVERAGE AGGREGATE VELOCITIES DEDUCED BY THE FACTORS.

	TEST.				
	G	*H*	*I*	*K*	*L*
Aggregate velocities in head-race, in feet per second	269.9	269.5	269.9	263.4	267.5
Distribution factor (tests of 1911)	0.00981	0.00981	0.00981	0.00981	0.00981
Mean velocity in head-race, in feet per second	2.648	2.647	2.648	2.583	2.623
Area of cross-section, in square feet	625.8	625.8	625.2	623.9	626.0
Discharge of meter section, in cubic feet per second	1 656	1 655	1 655	1 611	1 641
Discharge of centrifugal pump, in cubic feet per second	2	2	2	2	2
Correction due to rack error, in cubic feet per second	5	5	5	5	5
Total discharge, in cubic feet per second.	1 663	1 662	1 662	1 618	1 648

131.—Discharge in Tests N, O, P, Q, and R.—As in the preceding section, the discharges for Tests *N, O, P, Q,* and *R* can be computed from the aggregates of velocity for these tests. Table 88 gives the aggregates for the four systems of factors deduced. The remainder of the calculations will be clearly understood without further remark.

132.—Discharge in Tests S, T, U, and V.—Tests *S, T, U,* and *V* have been computed in the same manner. The final results are shown in Tables 90 and 91.

133.—Comparison of Discharges by Current Meter with Those by Chemical Method.—In making this comparison it must be understood that the tests in which both methods were used were the earlier and less reliable ones, in which there is considerably more uncertainty than in the later (numbered tests following Test 25) series. The

TABLE 88.—AGGREGATES OF VELOCITY AT THE 100 UNIFORMLY DISTRIBUTED POINTS IN THE HEAD-RACE OF UNIT NO. 13 FOR TESTS *N, O, P, Q,* AND *R*. THESE TESTS WERE NEARLY IDENTICAL AS TO CONDITIONS OF OPERATION, BUT ARE NOT SO UNIFORM IN THIS RESPECT AS *G, H, I, K,* AND *L*.

NATURE OF FACTORS USED FOR CORRECTING THE METER RECORDS.			TEST.				
From tests:	Meters compared by:	Common area and discharge.	*N*	*O*	*P*	*Q*	*R*
G, H, I, K, L..	Positions.....	{ 625 sq. ft..... { 1 660 cu. ft. per sec....	297.9	290.7	288.2	301.5	293.2
G, H, I, K, L..	Horizontals...	Not reduced.....	294.1	295.1	292.1	294.3	293.4
N, O, P, Q, R.	Positions.....	{ 619 sq. ft..... { 1 780 cu. ft. per sec....	297.6	295.0	293.1	296.0	293.3
N, O, P, Q, R.	Horizontals...	Not reduced.....	294.5	295.5	292.7	294.7	294.0
Average values....			296.0	294.1	291.5	296.6	293.5

TABLE 89.—DISCHARGE IN TESTS *N, O, P, Q,* AND *R*, BASED ON AVERAGE AGGREGATE VELOCITIES DEDUCED BY THE FACTORS.

	TEST.				
	N	*O*	*P*	*Q*	*R*
Aggregate velocity in head-race, in feet per second.....	296.0	294.1	291.5	296.6	293.5
Distribution factor (tests of 1911).....	0.00981	0.00981	0.00981	0.00981	0.00981
Mean velocity in head-race, in feet per second.....	2.903	2.885	2.858	2.908	2.878
Area of cross-section, in square feet.....	612.9	615.9	622.0	620.8	622.0
Discharge at meter section, in cubic feet per second.....	1 778+	1 776	1 777+	1 805	1 790
Discharge of centrifugal pump, in cubic feet per second.....	2	2	0	0	0
Correction due to rack error, in cubic feet per second.....	5	5	5	5	5
Total discharge.....	1 785	1 783	1 782	1 810	1 795

power was measured only by the switch-board instruments, and has been corrected as nearly as may be, but is probably not to be relied on to within 1 per cent. Hence, it is scarcely possible to state in all cases whether the current meters or the chemical method is the more accurate.

TABLE 90.—AGGREGATE OF VELOCITY AT THE 100 UNIFORMLY DISTRIBUTED POINTS IN THE HEAD-RACE OF UNIT No. 13 FOR TESTS *S*, *T*, *U*, AND *V*. THESE TESTS WERE RUN UNDER CONDITIONS WHICH ARE VARIABLE FROM TEST TO TEST.

NATURE OF FACTORS USED FOR CORRECTING THE METER RECORD.			TEST.			
From tests:	Meters compared by:	Common area and discharge.	*S*	*T*	*U*	*V*
G, *H*, *I*, *K*, *L*	Positions.	625 sq. ft. 1 660 cu. ft. per sec.	277.4	262.8	249.4	257.9
G, *H*, *I*, *K*, *L*	Horizontals.	Not reduced.	274.2	266.5	253.1	251.8
N, *O*, *P*, *Q*, *R*	Positions.	619 sq. ft. 1 780 cu. ft. per sec.	277.7	266.7	254.0	253.5
N, *O*, *P*, *Q*, *R*	Horizontals.	Not reduced.	274.6	267.0	253.6	252.5
Average value.....			276.0	265.8	252.5	253.9

TABLE 91.—DISCHARGE IN TESTS *S*, *T*, *U*, AND *V*, BASED ON AVERAGE AGGREGATE VELOCITIES DEDUCED BY THE FACTORS.

	TEST.			
	S	*T*	*U*	*V*
Aggregate velocity in head-race, in feet per second....	276.0	265.8	252.5	253.9
Distribution factor (tests of 1911).....	0.00981	0.00981	0.00981	0.00981
Mean velocity in head-race, in feet per second.....	270.6	260.7	247.6	248.9
Area of cross-section, in square feet.....	617.3	622.0	605.3	602.0
Discharge at meter section, in cubic feet per second....	1 669	1 621	1 498	1 498
Discharge of centrifugal pump, in cubic feet per second.	2	2	2	2
Correction due to rack error, in cubic feet per second.	5	5	5	5
Total discharge, in cubic feet per second.....	1 676	1 628	1 505	1 505

In particular, Tests *T*, *U*, and *V* are very doubtful as to the chemical method, as it is practically certain that an error was made in the laboratory when titrating the salt-solution samples of Tests *T* and *U*, and leaking pipes affected the results of Test *V*.

It appears that at this time the chemists had not succeeded in titrating all the samples of each test on the same day. Frequently, the salt-solution samples were titrated on the day following the titration of the remainder of the samples, and, in the case of Tests *T* and *U*, these salt-solution samples undoubtedly became interchanged, so as to cause Test *T* to give too great a discharge and Test *U* to give too small a discharge, each by several per cent.

In the case of Test *V*, Sample 22 titrated about 3 c.c. lower than it should, thus pointing to a leak in the corresponding pipe.

By interchanging the values of *v* as used in Tests *T* and *U*, Table 50, the discharges for these tests can be shown to be about 1 662 and 1 513 cu. ft. per sec., much more likely values than those given in the table. If Tests *T* and *U* are to enter into the comparison, these values should be used.

In the case of Test *V*, it is doubtful whether any improvement can be made in the calculation, further than to state that the discharge computed in Table 50 is too large, owing to the dilution of tail-water samples by reason of a leak. To compare methods, therefore, it will be advisable to omit Test *V* altogether.

Of course, the practice of titrating the salt-solution samples at any other than the time at which the remaining samples of the same test are treated is contrary to the principles of group titrations, fully discussed in Section 48, Part II.

The comparison of the discharge by the two methods is given in Table 92.

TABLE 92.—COMPARISON OF DISCHARGES DETERMINED BY CURRENT METERS WITH THOSE DETERMINED BY THE CHEMICAL METHOD.

Test.	Discharge determined by:		Remarks.
	Current meter.	Chemical method.	
G	1 663	1 662	
H	1 662	1 662	The figures in parentheses are corrected from the erroneous results of Tests *T* and *U*, as explained in note at foot of Table 50. They are the only figures which should be used in these comparisons.
I	1 662	1 666	
K	1 618*	1 683	* Meter in fifth position appears to run slower than usual.
L	1 648	1 647	
N	1 785	1 777	
O	1 783	1 768	
S	1 676	1 640	
T	1 628	1 746 (1 662)	Meter rack jammed, delaying meter test, which ended 17 min. after chemical test.
U	1 505	1 440 (1 513)	
V	1 505	1 553	Chemical test untrustworthy. Sample 22 indicates a leak in sampling pipe.

Omitting Test *V*, the total discharges by current meter and chemical method are 16 630 and 16 680 cu. ft. per sec., respectively, results differing by 50 cu. ft., which is 0.3%, on the average, during the tests.

The close agreement between the two methods for several of the tests is more or less of a coincidence, but is significant of the fact that there are no large systematic errors in either method.

Of course, it will be understood that the current-meter computations would have been much simpler had the meter rack been arranged so that direct comparisons of two meters of different types could be made in one and the same test, rather than to have to resort to comparisons in different tests. However, the work is more interesting in its present form.

Before closing, it must be repeated that the tests in which comparisons of the results by meters and chemicals can be made are by far the least reliable of any, excepting possibly a few where leaks in the sampling pipes vitiated the results altogether. Such tests are limited to a few in the series numbered from 1 to 25.

For much valuable advice and assistance of various kinds the writer is greatly indebted to Messrs. T. J. Bostwick, I. G. Calderwood, John R. Freeman, P. H. Falter, E. E. Haskell, George K. Herzog, J. H. James, D. J. Jones, H. W. King, Marks Neidle, H. C. Sadler, I. B. Smith, E. W. Schoder, W. M. White, Gardner S. Williams, I. W. Wilson, A. S. Yount; and the engineers and chemists more or less directly connected with the tests: Messrs. C. E. Andrews, G. H. Bancroft, J. Earle Brennan, J. B. Campbell, Clyde L. Frear, Y. Habeshian, F. G. Hess, J. G. Hinch, J. A. Lahey, W. A. Mudge, M. H. Montross, F. Nagler, A. J. Salathe, Arthur Schweier, C. M. Scudder, G. H. Spencer, M. L. Stack, K. T. Stearns, Thomas Sullivan, C. S. Thayer, D. C. Walser, W. H. Wagner, R. E. Ward, R. B. Whalen, A. L. Williams, and A. E. Wood.

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DISCUSSION

Mr. Kershaw.

G. BERTRAM KERSHAW,* M. AM. SOC. C. E. (by letter).—The writer has read Mr. Groat's paper with great interest. With respect to the word, "Chemi-hydrometry", as the author points out, this is a hybrid word derived from the Latin and Greek languages. As salt alone is used for mixing with the water, the writer would suggest the word, halo-hydrometry, derived from the Greek word ἁλς (hals) meaning salt. We already have the term, isohales, or lines of equal salinity, derived from ἴσος (equal) and ἁλς (salt). Halo-hydraulics would not be suitable, as the word ἀυλος signifies a pipe.

Will Mr. Groat kindly explain briefly the "moving-screen" method of measuring large volumes of flowing water, mentioned on page 952. The writer would also like to know whether the salt caused any appreciable frothing of the water when passing through the turbines, and whether there was any tendency of the meters to "short-circuit"?

Has Mr. Groat tried the use of coloring matter in the water in lieu of salt, for example, fluorescin; and does he consider a colorimetric method feasible?

With regard to the "shaking of samples", mentioned on page 1106, it was found necessary, during the taking of some thousands of samples of sewage, tank liquor, and effluent, to adopt a uniform method in order to secure a fairly complete mixture. The individual sample bottles were filled absolutely full, avoiding air bubbles as far as practicable. Actual shaking was found to be far less effective than quietly inverting each bottle for about 5 sec., and then reversing it, the operation being repeated ten times in succession. The absence of an air bubble in this case lengthened the period required for satisfactory mixing.

The writer would be much obliged if Mr. Groat would give the names of the makers of the Ott and Haskell meters; he has used the small Price meter for many years, and has found it to be a most excellent instrument. He has also used the Ekman meter, which is provided with an ingenious compass arrangement whereby the angle made by the head of the meter with magnetic north can be ascertained at great depths. Generally speaking, current meters manufactured in England leave much to be desired.

Mr. Fuller.

GEORGE W. FULLER,† M. AM. SOC. C. E.—It occurs to the speaker that there is one phase of this subject which no doubt has been adequately considered by the author, but which is mentioned in the hope that it will lead to some discussion. Reference is made to the diffusion in water of solutions of chemicals such as salt.

---

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Mr. Fuller.

Some years ago, when the speaker was connected with the Lawrence Experiment Station of the Massachusetts State Board of Health, solutions of salt were used to measure the actual period of flow of liquids through sedimentation tanks. The purpose was to ascertain the actual detention period of the liquid in the tank, as compared with the theoretical period obtained by dividing the capacity of the tank by the actual flow into and through it. By the use of salt applied to the entering water, and by tests of the effluent to determine how quickly the salt actually passed through the tank, information was obtained as to the extent which short-circuiting and other factors cause the actual detention period for part of the water to be less than the average or theoretical detention period.

Solutions of salt were also used at Lawrence to determine the water content, that is, the quantity of water held by capillarity in intermittent sand filters, in connection with studies to ascertain the best rate at which to operate the filters. Here, too, solutions of salt were added to the influent, and its rate of travel through the filter was determined by the tests of the effluent, in order to note the increases in salt content above the normal.

At many places, in recent years, in connection with the purification of water and sewage, use has been made of salt solutions and of aniline dyes to develop information along the lines just stated. Obviously, this use of salt solutions is somewhat different from that described by the author. In fact, it deals particularly with liquids moving at an extremely low velocity, whereas relatively high velocities would be used in a head-race in connection with tests of a hydro-electric plant.

However, it may be of some interest to mention that, at Lawrence, the diffusion of the solutions of salt proved to be a bothersome factor; in fact, some salt was found at the outlet of the tanks and filters at an interval so short after its application to the influent as to point most strikingly to the influence of diffusion. It is very likely that the use of salt for the purpose described so interestingly by the author would not involve similar complications, notwithstanding the fact that diffusion is a particularly disturbing element when applied under some conditions, as at Lawrence.

Mr. Peaslee.

W. D. PEASLEE,* ESQ. (by letter).—This classical paper is a very welcome and timely contribution to the literature of this subject, and Mr. Groat's masterly treatment is not a surprise to those who are familiar with his professional work.

However, the writer fears that the very mastery shown by this paper will cause the average engineer to regard what Mr. Groat so aptly terms "Chemi-Hydrometry" with a certain degree of misgiving,

* Dept. of Elec. Eng., Oregon Agricultural Coll., Portland, Ore.

Mr. Peaslee.

in that it would seem, on account of expense and intricacy, to be applicable only to those particular cases wherein the magnitudes and money involved warrant such a precision study as described. Although the degree of research and precision outlined in the paper is, of course, necessary in a test of the magnitude of this one, there is no limit to the field of this method when properly applied.

The writer has been working on this subject for more than 2 years, and although, on account of the high heads and small discharges common in the West, occasion has never arisen demanding the measurement of discharges greater than 300 sec-ft., a method has been developed* wherein several features have been worked out tending toward a greater simplicity of procedure, and sacrificing to only a small degree the accuracy secured by Mr. Groat.

The measurement and maintenance of the dosing rate is secured with the equipment shown by Fig. 40. Various sizes of nozzles, to fit the lower end of the pipe, are made, and the equipment can thus be calibrated for several different dosing rates, maintaining, with the hook-gauge, a constant head over the nozzle. As is well known, a properly designed nozzle orifice is one of the most accurate methods of determining small discharges. With this equipment the stop-watch error is almost entirely eliminated, as the calibration can be made over a long period of time. Also, as the calibration can be made with the dosing solution used, the effect of temperature and therefore of density can be eliminated. It has been found feasible to hold and determine the dosing rate within a total error of 0.1 of 1 per cent.

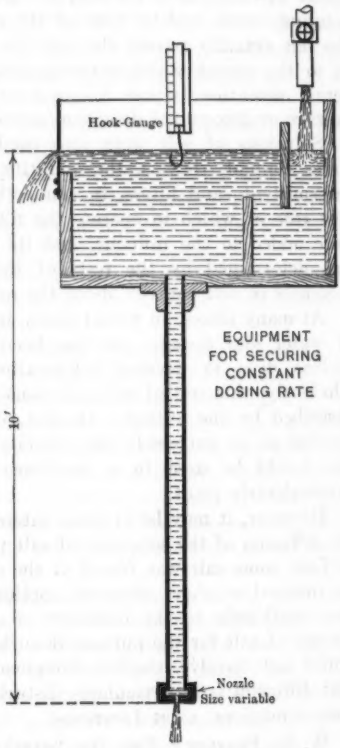


FIG. 40.

* *Journal of Electricity, Power and Gas*, August 21st and 28th, 1915.

Mr. Peaslee.

This equipment has now been in use for several months, in the following rather interesting way: A Central Station Company, carrying a rather large rural pumping load, has equipped an automobile with this apparatus, a quantity of salt, sample jars, electric meters, and a large tank. At certain intervals, two men with this outfit make the rounds of every pump on the company's lines and make discharge measurements of the pumps by this method, and at the same time take electrical input readings. The samples taken are sealed in jars and, with the meter readings, are sent to the chemist at the main office, where they are analyzed. In this way many very valuable data are secured at very small expense to the Company and with no trouble to the pump men. The capacities of the pumps range from 40 to 4 000 gal. per hour, and these two men, now that they are used to the work, can set up and run a test in a remarkably short time. The writer feels sure that the error in the results from these tests is 0.5 of 1% or less.

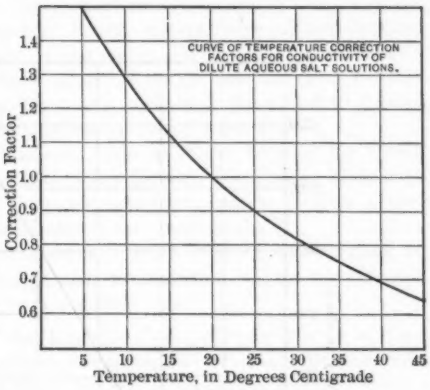


FIG. 41.

The point the writer wishes to bring out is that, especially for small discharges, it is possible, by the sacrifice of a very small degree of the accuracy of Mr. Groat's work, to have a method which will not appear to the average engineer as formidable as Mr. Groat's paper would suggest; and is, at the same time, cheaper than almost any other determination, and more accurate than most test conditions warrant.

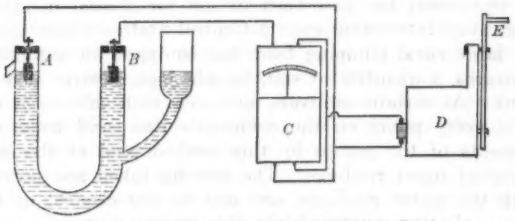
As there is only a very brief reference to the electric method of determining the salt content of samples, the writer will add a short description of some work he has done, using this method.

The following factors enter into and must be taken account of in any method involving the electrical determination of the salt content of samples:

1.—Temperature has a distinct and accurately known effect on the electrical conductivity of any saline solution. The curve of Fig. 41 shows this variation as it has been determined experimentally, verified by mathematical computation based on chemical theory.

2.—There is a counter electro-motive force of polarization that must be corrected for or taken into account in the calibration of the

Mr. Peaslee.



APPARATUS FOR ELECTROLYTIC DETERMINATION.

FIG. 42.

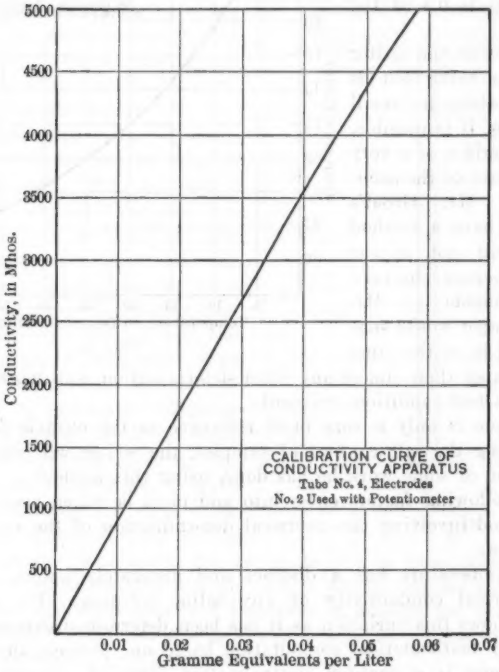


FIG. 43.

apparatus. For weak salt (NaCl) solutions and bright platinum electrodes, this is constant and a little less than 2 volts. Mr. Peaslee.

3.—Means must be provided for thorough cleaning of the apparatus, in order to maintain the integrity of the different samples.

4.—The formation of gas bubbles on electrodes must be prevented, or account must be taken of their effect.

The equipment shown by Fig. 42 has been developed and has proved successful when determinations on the samples within 0.1 of 1% are satisfactory, and further work is being done toward a refinement of this method. In this figure, *A* and *B* are bright platinum electrodes of special design to prevent bubble formation; *C* is a conductivity meter; and *D* is a direct-current generator of suitable design. The whole equipment can be made very compact and portable.

It is calibrated with known solutions, and a curve, such as Fig. 43, is obtained which takes into consideration the variations in cross-sections of the tube, the counter electro-motive force of polarization, and individual characteristics of the meter. With this equipment and the method of special dilutions, much of the tedious titration work is avoided, and the experience of a year's work leads the writer to depend on this method and equipment to within 0.1 of 1 per cent.

The field of this method is very wide, as it can be used for cheap, accurate tests on small discharges, up to, say, 500 sec-ft., and, with a corresponding increase in expense and care, can be used as a precision method on discharges of almost any magnitude.

The engineering fraternity owes a large debt of appreciation to Mr. Groat for his remarkably able presentation of the theory and practice of this method in such complete form.

W. L. Waters,* Esq. (by letter).—This paper covers in a very complete and thorough manner the technique of conducting an efficiency test on hydraulic turbines by the salt-solution method, but, in view of the great detail of the tests, the writer thinks that Mr. Groat could, with advantage, publish some additional figures in connection with the readings. Mr. Waters.

The principle of this test with a salt solution is that brine added to the water in the forebay becomes uniformly mixed with the whole water supply of the turbine. It is evident that, if the efficiency of the forebay and turbine were 100%, there would be no eddies in the water to enable the brine to mix and form a solution of uniform density with the whole of the water supply. The brine, when added in the forebay, would simply move in stream lines parallel to those of the water supply, so that it would not mix.

In each test, Mr. Groat takes eighteen samples of the tail-race water, and it is probable that a comparison of the figures obtained

* Pittsburgh, Pa.

Mr. Waters. for each of these eighteen samples would give a very fair idea of the extent to which the brine solution becomes uniformly mixed with the forebay water in the average high-efficiency modern turbine.

If it is found that the brine does not mix very satisfactorily in an efficient turbine, it would indicate that difficulties may be expected in applying this otherwise very attractive test to the case of turbines with a very high efficiency. It is probable that the figures obtained by Mr. Groat will enable a fairly definite opinion to be formed as to the degree of mixing obtained, if the results obtained in each test of the various eighteen sample tubes in the tail-race are given. On this point Mr. Groat could add greatly to the value of the paper if he would give some additional test figures.

Mr. Horton. ROBERT E. HORTON,* M. AM. SOC. C. E. (by letter).—Any one who has undertaken an original investigation in a new field must realize the enormous amount of painstaking and careful labor which has been involved in the preparation of Mr. Groat's paper. On looking it through, it appears that he has presented the original data fully, freely, and without reserve, thereby setting an example which may be properly followed by others offering the results of original investigations. As a rule, even though errors may be found, the results of such an investigation are likely to inspire greater confidence, and are certain to be more useful in assisting future progress in the work if presented in full, as has been done here.

The author suggests the use of the term "chemi-hydrometry" to describe the measurement of the volume of flowing water by the introduction and subsequent detection of chemicals. As he suggests, this name is not very satisfactory. The terms "hydrometry" and "hygrometry" have been pre-empted, unfortunately, to describe somewhat limited special aspects of the measurement of water. There is great need for some term to cover all classes of measuring water, itself an art of far-reaching importance. In fact, there is need for a general revision of hydraulic terminology. The term "hydrography", sometimes used to describe the measurement of water, should be limited in its application to the mapping of waters, giving it a meaning somewhat analogous to the words "topography" or "geography" in relation to the mapping of lands. Hydrology, in its broad sense, covers every aspect of the science and phenomena of waters, but should generally be used in the more restricted sense of the science covering the laws and phenomena of occurrence and distribution of waters in their natural state. It would thus include the phenomena of precipitation, evaporation, yield and regimen of streams, and the flow of groundwater. In this sense, the word "hydrology" is akin to the word "geology" in relation to the solid materials of the earth.

* Albany, N. Y.

Mr. Horton.

The writer suggests the use of the term "aquametric", as being both available and expressive, to describe matters relating to the measurement of water in general. Thus an aquametric survey would be a survey for the purpose of determining the quantities of water in a region. In the present instance, as the measurement is accomplished by the use of salt, the process might be denominated a "sal-aquametric" method.

Regarding the current meter, the writer believes it is a much abused instrument, and, at the same time, its value and utility are often greatly over-estimated. Under suitable conditions, it may quite certainly give results of a single measurement of a stream accurate within less than 1 per cent. This would require that the channel be of sharply-outlined, definite, and well-determined cross-section, and that the stream should be flowing below the critical velocity, so that stream-line, or filamentous, flow takes place. The writer concluded some years ago that there was danger in accepting current-meter ratings made in still water as being always applicable to flowing water. Until recently, the reasons for this do not appear to have been set forth satisfactorily. A recent book* shows logically and conclusively that errors such as have been referred to might exist, and that their cause rests essentially in the difference in fluid resistance under conditions where the fluid is static and the object is moving, as compared with the resistance where the object is quiescent and the fluid is moving in a non-uniform or sinuous manner. Experiments on the flow in pipes show that the critical velocity at which motion changes from the stream-line system to the vortical flow is not the same when the fluid velocity is gradually increased, as it is when the velocity is decreased from a higher to a lower limit.

As illustrating the conditions under which highly accurate results may be obtained with a current meter, Fig. 44 is presented, showing the velocity contours in a cross-section of a flume, of smooth, planed plank, specially constructed for the purpose of measuring the stream, the velocities being taken with a current meter suspended from a framework in such a manner that the exact position of the center of the meter could be determined with a high degree of accuracy.

As an illustration of the uncertainties of current-meter work unless proper precautions are taken, the writer recalls an instance which occurred a number of years ago. A dam was being used as a weir for the purpose of measuring the yield of a stream on which a power plant was to be constructed. It was known that there was considerable leakage through the dam, and an attempt was made to determine the volume of this leakge in the early winter, at a time when there was some water flowing over the crest of the dam, and it was impossible to draw the water down to crest level. There was a masonry aqueduct

* "The Motion of Liquids," by R. De Villamil.

Mr. Horton.

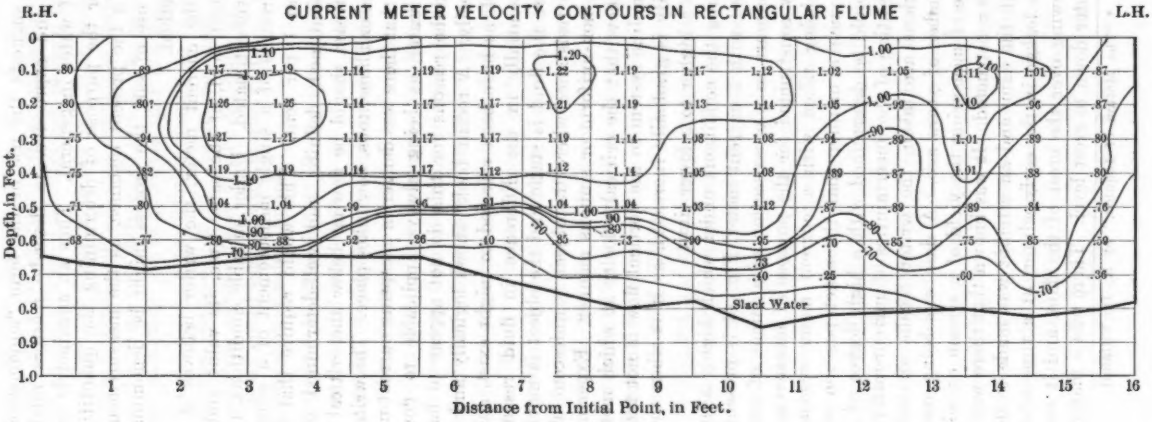


FIG. 44.

Mr. R. Horton.

across the stream a few hundred feet below the dam. This aqueduct had about a dozen archways, and was constructed with a parapet over which one could look vertically down to the water surface, but it was impossible from any position on the aqueduct to look under the archways. The idea was to make a current-meter measurement of the flow under this aqueduct, subtract from the measured flow the calculated flow over the spillway, the remainder being the indicated leakage through the dam. The writer is informed that the measurements were made by current meter in the usual manner, and that from surface indications there was apparently a uniform current at right angles to the aqueduct passing under each archway. The stream was probably turbid at the time, so that it would have been impossible to observe the position of the submerged meter had an effort been made to do so. These measurements indicated an apparent leakage through the dam of 315 cu. ft. per sec., which the writer is informed was accepted as a basis for estimating the low-water flow of the stream in connection with the proposed power development. Being suspicious of these results, the writer undertook to make an independent determination of the leakage through the dam in the following summer. Examination of the aqueduct showed that most of the leakage came down the right-hand side of the stream; reaching the up-stream side of the aqueduct, the current was deflected across the stream, and the flow finally passed out under the left-hand arch. The deflection of the current was caused by the existence of old falsework under all the archways except the one on the left-hand side of the stream. At the time the writer's observations were made, the water was drawn down to the crest of the dam, but further observations showed that, when there was a sufficient volume of water wasting over the dam, the water would spill over the obstructions under the archways in such a manner that there was an appreciable surface current directly down stream, although the main flow was across the stream channel. Apparently, the current meter had been headed across the stream, and the same water had been measured over and over again, as careful measurements by wading, where the leakage through the dam was concentrated in a single channel, indicated the actual leakage at crest level to be only from 35 to 45 cu. ft. per sec.

The power development based on the erroneous measurements was a failure. The writer had nothing to do with the original measurements, and did not receive any very cordial thanks for calling attention to the error, as the owners of the proposed power development were just at that time attempting to issue additional bonds to complete their construction.

Sal-aquametric turbine testing is evidently laborious, requires a great refinement of methods, and is costly. There appear to be many situations where it would be difficult to apply this method in

Mr. Horton. practice. The same may be said of any other method of water measurement in particular cases. Therefore, no single aquametric method should be either universally condemned or accepted to the exclusion of all others.

Mr. Wiggin. THOMAS H. WIGGIN,* M. AM. SOC. C. E.—The author is to be praised for the care with which he has worked out all sources of error. Probably some of these errors would never require attention in ordinary tests; but it is well to know the sources of even small ones and to have them explained so that they may be avoided. The speaker has had something to do with testing machines, and knows how easy it is for errors to be overlooked. For example, on so simple an apparatus as a mercury gauge for showing heights of water, there are several sources of error which are commonly neglected, and could amount to quite a percentage. The speaker remembers one pumping engine test where unusual care was taken, there being a good-sized bonus in question, but in which one source of error which was not taken into account amounted to several thousand dollars.

The question intrudes itself, however, as to whether it is ever justifiable to write a specification in which the degree of accuracy indicated by the author is attempted; in other words, whether the cost of testing is not more perhaps than the amount involved, and whether still further refinement might not prove the apparent precision to be in part fictitious. Furthermore, is the quantity to be measured—namely, the efficiency of the turbine—sufficiently constant for a considerable period of time to justify such precision? It would be of interest if the author could state whether, after a week, 10 days, or a month, this same machine, tested to the same accuracy, would show the same efficiency. Perhaps the author can tell of tests conducted at intervals on the same turbine which would show whether the machines changed.

A water-works engineer is glad to get a Venturi meter in his line which will enable flows to be measured to within 1% or so; but it is not always possible, and occasions come to the speaker's mind in which the author's method of measuring flows will be indispensable, particularly when it is reduced to its simplest terms. The speaker presumes that the actual performance of such tests, now that the author's exhaustive investigation has been completed, will be much less formidable than is indicated by an examination of the paper and the illustrations.

The cost of these investigations must have been considerable, and the author and others who have generously contributed the results to the Engineering Profession deserve thanks and appreciation. It would be of interest if the author could state, roughly, the probable cost of a future test conducted with the care indicated, now that the method has been established; also, for purposes of comparison, rough figures

* New York City.

for the commercial value of small increases in turbine efficiency, justifying extreme precision in such tests. Mr. Wiggin.

W. S. RICHMOND,* M. AM. SOC. C. E. (by letter).—The excellent use which Mr. Groat has made of his exceptional opportunity to carry on experiments in hydraulic measurements, while performing important hydro-electric tests, is highly commendable. It would seem that every item of the work had been considered, experimented upon, and discussed, in so far as it was at all pertinent. The paper is a very valuable addition to hydraulic literature. Mr. Richmond.

Mr. Groat remarks that in some cases, where apparently excellent current-meter work has been done, the results are chargeable to art as much as to science. With this idea the writer agrees, and he believes the fact is not objectionable, as he understands the author's remark to indicate. Long familiarity with the Haskell meter has convinced the writer that its use and care involve as much of art as of science in those instances where it is made to give the best account of itself, exactly as is the case in the delicate manipulation of a transit or a ruling-pen.

The writer had the great pleasure of spending an hour with Mr. Groat at Massena in 1914 while the tests were in progress. He did not see any current-meter measurements being made, but did see the gauging sections, and was permitted to examine the current meters. The writer then felt that the author had before him a difficult task in both the chemical and the current-meter gaugings. On the one hand, conditions of low head, large volume of flow, and short distance between dosing and sampling sections appeared disadvantageous for the chemical gauging; and, on the other hand, the sections necessarily chosen for the current-meter observations were obviously poor. Moreover, the chemical gauging was pioneer work. So far as the writer's knowledge goes, the only other instance of chemical gauging of importance in connection with the testing of hydro-electric generating units in America is that by Mr. Abraham Streiff for the Fargo Engineering Company, at the Croton power-house, on the Muskegon River, in May, 1914. The agreement of discharge measurements by chemical and by current-meter methods there is reported to have been 1.3 per cent. Great credit is due to Mr. Groat for obtaining such accuracy as he has in the chemical gauging, and he is to be congratulated on the excellent apparent agreement of results by chemical and current-meter methods.

The author states that the rigid method of support is very unfavorable to the Haskell meter, and that it is favorable to the Price meter. This is strikingly shown in the results of the experiments performed in the Naval Tank at Ann Arbor. The large errors introduced in

* Detroit, Mich.

Mr. Richmond.

the case of the Haskell meter are surprising, and indicate clearly that this method of support should not be used if it can be avoided. During the work at Massena in 1911 the meters were provided with tails, and were supported by rods or cables in such a manner as to be free to swing both horizontally and vertically. The results then obtained at very poor sections showed comparatively small errors in the Haskell meter, and rather large errors in the Price meter. The conclusions then reached were in general harmony with those of the writer, who has used only meters provided with tails and suspended by cables. In this case the angle which the meter makes with the perpendicular to the section at the same point must be measured with sufficient accuracy, and the proper correction applied where necessary. That none may be misled, it seems proper to point out that no errors will result when a Haskell meter, provided with a tail and suspended by a cable, is used carefully at a really good section. That the head-race section at Massena was very poor is shown with sufficient clearness by those observations in the double vertical, No. 5, where the divergence between Haskell and Price meters (assuming no change in still-water ratings) indicates a transverse component of velocity approximately one-half as great as the axial component.

The writer cannot agree with Mr. Groat that "under the most favorable circumstances" current meters may indicate velocities in error by 2 to 8% or even more, when an ordinary still-water rating is assumed to apply. Haskell and Price meters used by the writer have shown no such errors, except when used under unfavorable circumstances, and the error of the Haskell meter has been small under very unfavorable conditions. The writer has used the small Price meter when it over-registered by 34%, and the Haskell meter when it under-registered by 7%, but the conditions had been made exceedingly unfavorable purposely. The maximum error observed by the writer while actually gauging stream flow was approximately 11% for a large Price meter, well rated and in good condition, but suspended from a catamaran which was pitching in a sea so as to give the meter a vertical motion of 3 or 4 ft. amplitude. Under similar circumstances, the error of the Haskell meter has been too small to be determinate. Either meter will frequently give errors of from 1 to 2% in a single short-run measurement, but, under favorable conditions, the mean of several measurements will be the true velocity that obtained, within a fraction of 1 per cent. The writer firmly believes that a B-type Haskell meter, in fine condition, skillfully used, in smoothly flowing, reasonably clear water such as obtains at well-chosen gauging sections on the large streams of the Great Lakes system, may be relied on to indicate the true velocity within a fraction of 1% when its still-water rating is applied, provided the velocity is

Mr. Richmond.

more than 1½ ft. per sec., and the mean of several runs is taken. A Haskell meter frequently will not remain in such a condition long, without attention, and it is necessary to use another meter, or some other equally accurate device, simultaneously, in order that instant knowledge may be had that something is wrong, so that the trouble may be remedied at once. In most of the work of the U. S. Lake Survey, three or more meters have been used on any piece of gauging, two meters being operated simultaneously at the same elevation a few feet apart. By changing meters frequently, and pairing them in as many combinations as possible, it is usually easy to tell which, if any, have changed their ratings, either temporarily or permanently. This has been called the "two-meter method", and, the writer believes, was devised by F. C. Shenehon, M. Am. Soc. C. E., formerly of the Lake Survey, and now of the University of Minnesota. In all Lake Survey work the Haskell meter has been used with tails on the body of the meter and on the weight, the meter being suspended on a flexible cable. This method the writer heartily recommends.

That the still-water rating is applicable without correction in the case of the Haskell meter when used at a good section has been demonstrated in several different ways by the work of the Lake Survey. Perhaps the best test was that made in 1906 in the Detroit River by Mr. Shenehon, under the general supervision of Mr. Haskell. In these experiments, two very stable, light-draft, catamarans were anchored in the river in tandem, being held at a certain distance apart by wires. A Haskell meter was suspended 12 ft. from the side of each catamaran, and 1 ft. deep, the two meters being in the same current thread, one 205 ft. down stream from the other. Liquid blueing was injected into this current thread several feet up stream from the up-stream meter, and carefully timed over the distance between the meters, while the meters simultaneously recorded their revolutions electrically. Seventy-six such observations were made, the mean velocity being 2.862 ft. per sec. by the bluing, and 2.859 by the meters.*

Another proof is furnished by the simultaneous use of Haskell and Price meters in the St. Clair River in 1909. The writer was not then familiar with the experiments by C. H. Miller, M. Am. Soc. C. E., and was considerably chagrined at the time because the meters did not always agree. He soon noticed that the principal discrepancies occurred on those days when the observing catamaran had a vertical motion, due to waves on the river, and that the divergence between the meters increased with the vertical motion. It was found that the measurements by the Haskell meters, when related to the corresponding stage and slope of the river, satisfied very closely the equation expressing the mathematical relation of discharge to stage

* This test is described more at length in Senate Doc. 105, 62d Cong., 1st Sess., pp. 72–74.

Mr. Richmond.

and slope, and that the discharge measurements by the Price meters were too great. This equation had been derived from the results of a great many measurements, most of which had been made under smooth-water conditions, so that the equation itself contained no appreciable error from this cause. Furthermore, on comparatively calm days, simultaneous measurements by the Haskell and Price meters agreed within about 1 per cent. Mr. Groat has shown that in such case the true velocities lie between the values by the two meters. In 1910 the writer made oscillation experiments at a rating base and at one of the hydraulic sections, the results agreeing closely with those obtained by Mr. Groat at Massena in 1911, and showing that a much larger correction must be applied to the Price than to the Haskell meter indications. The Haskell meters, therefore, probably were correct within less than 0.5 per cent.

A third proof was obtained in 1914 at a gauging section on the St. Lawrence River, using a Pitot tube simultaneously with a Haskell meter. The Pitot tube was in general patterned after that recommended by W. M. White,* M. Am. Soc. C. E., and also used by Mr. Groat at Massena in 1911. This tube has been shown by both these gentlemen to give very accurate velocity indications in a current. The writer also made careful experiments confirming their opinions concerning the accuracy of the static orifice. Several sets of simultaneous observations with Haskell meters and the tube were then obtained in a very smoothly flowing current of about 4.2 ft. per sec. The mean result showed the meters in agreement with the tube within 0.4 per cent.

As an indication of the precision that may be obtained in current-meter gauging of large rivers, it may be stated that three separate sections on the St. Clair River, many miles apart, and of entirely different characteristics, showed mean agreements of discharge within 0.4% when compared through the relations of discharge to stage and slope. Similarly, three sections on the Niagara River showed an outside agreement of 1.3%, and two sections on the St. Lawrence River agreed within 0.4 per cent.

Mr. Groat.

BENJAMIN F. GROAT,† M. AM. SOC. C. E. (by letter).—Mr. Kershaw's suggestions as to the use of the term "halo-hydrometry" are worthy of consideration; but the term is narrower than chemi-hydrometry, and thus fails to include chemicals which are not salts—acids, for example. Chemi-hydrometry includes color, conductivity, or any other property which can be changed by the introduction of a chemical.

The moving screen has been used in Europe to some extent, and has been proved to be more accurate than any other method of measuring large volumes of flowing water, with the exception of chemi-hydrometry, which may be made as accurate as desired, with suffi-

* *Journal* of the Association of Engineering Societies, Vol. 27, August, 1901.

† Pittsburgh, Pa.

cient care and expense. A full explanation of the method may be found in a paper entitled "The Diaphragm Method for the Measurement of Water in Open Channels of Uniform Cross-Section",* by Carl Robert Weidner, Assoc. M. Am. Soc. C. E. Mr. Groat.

The method requires a diaphragm which fits the cross-section of a prismatic conduit, or flume, as closely as practicable. This screen traverses the length of the conduit on a carriage which rolls on wheels guided by a track. The volume described by the screen in any observed period of time is theoretically equal to the volume of water flowing in the conduit during the same interval, supposing the screen to move with the average velocity of the water.

Concerning the action of the salt on the feed-water, the salt, being in very dilute solution, caused no "frothing" of the water when passing through the turbines, and there was no short-circuiting of the meters due to its presence in the water, although attention was called to its action on the water rheostat when it was used in Test No. 105. (See Section 109, concerning the first series of tests of Unit No. 11.)

The writer has used color in a series of tests with very satisfactory results. Colors may be matched accurately with a colorimeter, but there is no colorimeter specially adapted to the requirements of chemi-hydrometry, the need of which is suggested at the close of Section 39. The principal objection to color is its cost.

The Ott meter is made by A. Ott, Kempten, Bavaria. The Haskell meter is made by E. E. Haskell, M. Am. Soc. C. E., Cornell University, Ithaca, N. Y. The deep-sea Haskell type gives the velocity and direction of flow at any depth.

Mr. Fuller describes very interesting tests to determine the detention period for liquids flowing through tanks, and to determine capillary volumes by means of salt solutions injected into the influent and measured in samples of the effluent. It is very apparent that diffusion may have a considerable effect on the results thus determined where the velocity of diffusion is large relative to the velocity of the water, or other liquid, into which the salt is injected.

In the case of turbine tests, however, the velocity of the water is relatively high, and the mixture is so nearly perfect that the effects of diffusion are small; in fact, diffusion assists the rate of mixing, and produces a more uniform mixture than could otherwise be obtained. When the mixture is perfect, there is no diffusion.

The writer does not share with Mr. Peaslee the fear that engineers will imagine that they must duplicate all the detail of the work which was undertaken in order to develop a theory and method for chemical testing. It is very easy to see, for example, that the chemical method may be applied with great simplicity to the test of a centrifugal pump, as described in Section 51. It will be observed, of course, that

* *Bulletin* of the University of Wisconsin, No. 672, 1914.

Mr. Groat. no evaporations are necessary in this and similar cases, and that the necessary chemical equipment consists of a comparatively small salt-solution tank connected to the suction of the pump, with means for measuring the weight or volume of the solution consumed, and a very simple outfit for taking and treating the samples from the discharge.

In the case of larger operations, the tests need not be any more complicated, except that where great accuracy is required with large volumes of water to be measured, it will be found advantageous, in the majority of cases, to resort to evaporation for the purpose of concentrating the samples. About the only real difference between the test of a turbine and the test of a pump lies in the magnitude of the operation, which varies to a less relative extent than the quantity of water measured. Nor is it expected that the reader will necessarily adopt the detail of any of the methods treated in this paper. Engineers are capable of planning their own tests. What they want is reliable theory and reliable information on which to base their calculations, estimates, and specifications, without the necessity of experimentation. It was this want that led to unusual efforts in the tests which form the subject of the paper, and the writer will indeed feel satisfied if it is considered that he has, in some measure, supplied the deficiency.

Mr. Peaslee's testing automobile is very interesting. There are, no doubt, many opportunities for the use of automobiles in transporting testing equipment from place to place, and it would seem that there is no reason why the titrations could not be performed on the spot by the men who execute the remainder of the test of any particular pump. With very little practice any engineer can make a titration to 0.1 or 0.2 of 1% by following, in a general manner, the directions in Sections 88 and 89. It should be remembered that the proper rate of draft for titration from an ordinary 75 or 100-c.c. burette is 8 or 10 c.c. per min.

Mr. Peaslee's description of his electrical equipment is very acceptable, as there are few papers, especially in English, on conductivity as applied to chemi-hydrometry. It might be remarked that with the application of the method of special dilutions, or balanced samples, Section 40, it will not be necessary to calibrate the instruments, as the procedure there described eliminates all the instrumental errors. In order to eliminate by special dilutions the effects of temperature, care must be taken to have each pair of tail-water and special-dilution samples at the same temperature. This follows as a corollary from the principle of balanced samples enunciated in the second paragraph of Section 12. One-tenth of a degree will affect the conductivity appreciably. With the exercise of care and judgment, the other errors, also, will be eliminated by a rigid application of the principle.

Mr. Waters asks for additional figures in connection with the readings, apparently referring to the readings for each of the eighteen tail-

race samples taken in each test. The writer has published these in full detail on pages 1182 to 1204, inclusive; and, in Section 50, one of the tests has been examined with respect to the distribution of salt among these eighteen samples. Mr. Groat.

The chemical data of each test have been given in full so that any one can make such study and analysis of them as he may desire, and the results of an investigation of this kind would be welcome. It is quite possible that there may be an error of computation in some of the discordant tests, which could be discovered in this manner. The writer would appreciate being informed of any such discovery.

Mr. Waters is correct in his theory that it should be relatively more difficult to obtain a perfect mixture with relatively high turbine, or pump, efficiency. Practically, the power required in mixing must be very small. Imagine, for example, a barrel of 200 liters of water with, say, 20 "stream lines" or "pencils" of salt solution uniformly distributed in vertical positions throughout the volume. Let a man mix this with a single vigorous double stroke of a large churn dasher, which is then quickly removed from the barrel. One can scarcely doubt that in a few seconds after removing the dasher the mixture will be near to perfection. If not in a few seconds, then certainly within a minute, about the time required for the mixture to pass through the turbine from the sprinkling pipes to the sampling pipes.

The energy exerted by the man can probably be represented by an average of 10 lb. acting 2 ft. downward and then 2 ft. upward, say, 40 ft-lb., or 40 ft-lb. is sufficient energy to cause 200 liters of water and salt to mix while it is flowing from the sprinkling pipes to the sampling pipes. To mix 50 000 liters, the turbine discharge per second, would require 250 times the energy above calculated, or 10 000 ft-lb. If this amount of energy is supplied each second, the entire discharge of the turbine will be thoroughly mixed before it reaches the sampling pumps, and it is easy to see that the power required is about $10\,000 \div 550 = 18.2$, or less than 20 h. p., to mix the discharge of a 6 000-h. p. turbine. In all probability, the eddies generated by the flow of water through the trash racks and against the sprinkler pipes in the head-race are sufficient to cause the necessary mixing of the salt with the feed-water, and the mixing will be aided by any diffusion which may occur.

Mr. Horton suggests the term "aquametric" as one descriptive of matters relating to the measurement of water in general. This term, like chemi-hydrometry, is hybrid, and does not appear to furnish a very euphoneous noun. Aquametry is constructed of Latin and Greek roots, the pure Greek synonym being, simply, hydrometry. It does not appear that there should be any serious objection to the broader use of the term hydrometry, even though usage has given it a more restricted meaning in relation to density and specific gravity.

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It was not the writer's intention to present primarily a paper on the current meter, but the methods used in correcting the indications of these instruments, and especially the composite ratings, were thought to be of sufficient interest to be presented in full.

In another series of tests it would be advisable to have readings by the two meters compared at each meter point. This would render the work of reduction much simpler, and be more satisfactory when it is desired to compute the discharge in each individual test. However, it will be observed, when treating all five of the tests in any series as though they were a single test, that the four methods of correcting the discharge give results which agree very closely, as may be seen in Table 85, where the difference between the maximum and minimum aggregates for Tests *G*, *H*, *I*, *K*, and *L*, as determined by the four methods, is only 8 in 1 340 or 0.6 per cent. That the agreement is not a mere chance may be demonstrated by comparing, in similar manner, the aggregates of velocity for Tests *N*, *O*, *P*, *Q*, *R*, as determined by the four methods, Table 93 giving the results for comparison.

TABLE 93.—AGGREGATE VELOCITY FOR TESTS *N*, *O*, *P*, *Q*, *R*, BY THE FOUR METHODS.

(Velocities, in feet per second.)

Factors based on tests.	Meters compared by:	Aggregate velocity.
G, *H*, *I*, *K*, *L*	Positions	1 471.5
G, *H*, *I*, *K*, *L*	Horizontals	1 469.0
N, *O*, *P*, *Q*, *R*	Positions	1 475.0
N, *O*, *P*, *Q*, *R*	Horizontals	1 471.4

In all probability, the individual tests, when computed by the four methods, would have compared about as well as those shown in Table 93, had it been possible to compare the records of meters at a given point in the same test. It will be observed that there are small systematic errors among the four methods. In this respect Table 85 seems to be quite consistent with Table 93.

The writer attributes the principal part of the deviations of current-meter ratings to the difference which exists in the condition of the water during the rating and at the time of the discharge measurement, and considers these deviations to be due only slightly to the fact that the meter is towed forward during the rating and held at a point during the discharge observations. In other words, he does not believe that there is any material difference in the hydraulic resistance of a liquid on a body, whether that body moves through the liquid or the liquid moves about the body, supposing the actual dynamic condition of the water, relative to the body, to be the same in the two cases.

Mr. Wiggin is very properly cautious as to the degree of accuracy of turbine tests, especially as to the possibility of apparent precision being in reality more or less fictitious or imaginary. When a whole series of tests is highly self-consistent, it may be safely concluded that the observations were made with a correspondingly high degree of precision as regards accidental errors. The theory of errors usually presented in treatises on probability, however, says nothing at all about systematic errors, which, indeed, are the errors most to be feared, as they are generally masked or otherwise hidden, so as to be easily overlooked. Mr. Groat.

As to the proper degree of accuracy to be required in turbine tests and the constancy of turbine efficiency, Mr. Wiggin and the writer seem to have somewhat divergent views. From numerous power capacity tests and a number of complete tests where the discharge was also measured by the writer at different times, turbine efficiencies have been found to remain remarkably constant when the machines have been kept in uniform condition and not allowed to become damaged. There seems to be no reason to believe that there are any material variations of this sort in actual practice where the condition of the machinery is well maintained. Good evidence to this effect may be found in the three series of tests on Unit No. 13. These tests extended over the period from July 22d to September 2d, inclusive, and no indication of variation in efficiency is observable. Turbines which were tested in 1911 still have the same capacity curves which were determined at that time. The writer is of the opinion that turbine testing, especially as to the measurement of the water, has been altogether too crude, and that the present requirements demand an increase in the degree of precision for the tests.

It would appear that a unit of power gained by an increase in efficiency is more valuable than a unit of power requiring an additional amount of construction. For it brings the same price on the market and requires no investment other than the cost of securing the increase in efficiency, which perhaps involves nothing more than the engineer's judgment in selecting the better of two designs, and thus is obtained without any increase of expense in operating the hydraulic part of the plant. Of course, there may be required a slight increase of cost and operating expenses for the electric part of the plant in the case of hydro-electric developments. Therefore, if a hydro-electric development costing $100 per h. p. is a paying proposition, and additional power can be had by an increase in efficiency, there is no reason why this additional power would not be well worth $100 per h. p. also, supposing that the market will absorb it without being affected. Perhaps it would be satisfactory to divide the gain due to increased efficiency between the purchaser and the turbine maker by letting the bonus be one-half of the development cost, due

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to the construction of the hydraulic part of the plant, including dams, canals, gates, flowage rights, etc. Great caution should be exercised in specifying turbine performance where power is to be developed by means of high efficiency. It would be very easy, indeed, to test turbines and pay a bonus at the rate of $100 per h. p., or one-half of this, only to find that operating conditions would seldom, or never, admit of working the machines under the conditions which secure this power. In all probability, specifications involving these large rates of bonus should not specify maximum turbine efficiency, but rather the minimum efficiency for a considerable range of probable operating conditions, such ranges of condition, for example, as might be specified by mentioning a definite area on a diagram similar to Plate LI.

There is no reason why chemical tests should be more expensive than tests of other kinds. In all probability, they can be made less expensive, especially for a given degree of accuracy. Turbine tests are more or less costly, because they are very complicated, no matter what method is adopted. It requires a considerable number of observers to obtain the data, particularly where the power is measured electrically. Moreover, a large number of tests are required where any attempt is made to cover the range of operating conditions. These remarks relate, of course, to very large power units. A small machine, such as a Pelton wheel or a centrifugal pump, can be tested chemically, with high precision, for a few dollars. The same is true of a water main, though large sizes may present greater difficulty.

The cost of turbine tests will depend on circumstances and on the methods and details adopted by the testing engineer. With the essential methods of testing well in mind, an engineer should be able to devise his tests and make accurate estimates of the cost. He need not be surprised, however, if he should find his estimate to be in the neighborhood of $5 000 for fifty or a hundred tests on a 10 000-h. p. turbine on a head of 50 ft., though circumstances might reduce this to $3 000, or even less. These costs are supposed to include the cost of electrical instruments and the engineering service necessary to conduct the tests and compute the results. The reason these figures seem high is because the tests are turbine tests, and not because they are chemical tests. Circumstances may run the cost higher still.

The writer remembers Mr. Richmond with pleasure. It is true that the meter sections which Mr. Richmond saw were unusually poor, and it is this fact that makes the meter work more valuable, for it shows what corrections are necessary under such conditions. The writer thinks, however, that the conditions for chemical work were favorable, and that there should not be too great a distance between the dosing and sampling stations. Otherwise, the eddies and dead water will require such a long time to fill with chemical that an ac-

curate test will be difficult or impracticable. In the case of large cross-sections, it is advisable to introduce the chemical from the sprinkling pipes on the down-stream side of the trash racks properly stopped sufficiently to raise a head of 8 or 10 in., thereby securing an almost uniform mixture at the outset. Mr. Groat.

The close agreement between some of the current-meter and chemical tests published in the paper is probably more or less accidental, but the writer is led to the belief that current meters can be used successfully in the manner described, so as to secure a degree of precision within 1%, if this was not actually secured in the present case.

There are two conclusions with which Mr. Richmond disagrees. The writer believes in rigid supports, properly designed, cupped, screw runners, and no tails or rudders, and he states on page 952:

"The uncertainty attending the measurement of large volumes of flowing water by methods heretofore used, under the most favorable circumstances * * * , is * * * , [for the current meter] 2 to 8%, and even higher."

In advocating rigid supports and no tails, the writer had in mind the measurement of turbulently flowing water in sections which, perhaps, might ordinarily be thought to be wholly unsuitable for a discharge measurement. That the discharge can be accurately thus measured in such sections by means of rigid supports and proper methods of correcting the meters seems to be accomplished in the tests, and this where the ordinary method would surely fail by a relatively large amount. Even under favorable conditions, the writer adheres to rigid supports and no tails, and advocates a new type of runner designed so that its record will always give the true integrated component of velocity perpendicular to the cross-section. There is plenty of evidence that the latter design may be effected without much experimentation.

The writer's statement concerning the accuracy of current-meter work under favorable circumstances refers to the accuracy of discharges, determined by the meters, and not to tests of the accuracy of the instruments. This is probably the cause of the misunderstanding. Ample authority for the statement can be found in the reports of the Lake Survey engineers. As an example, the interested reader may refer to the Annual Report of the Chief of Engineers on the Survey of the Northern and Northwestern Lakes for 1902, Appendix EEE. The report of F. C. Shenehon, M. Am. Soc. C. E., begins on page 2779, and shows a discharge curve of the St. Lawrence River with limits for errors immediately preceding page 2793. The limit of error is 2% either way, from the discharge curve. These are accidental errors. There is nothing to show that the systematic error

Mr. Groat. may not be larger. Opposite page 2826 will be found a plotting of a discharge curve by L. C. Sabin, M. Am. Soc. C. E., showing errors in the plotted points of several per cent. The report of W. Edward Wilson, M. Am. Soc. C. E., beginning on page 2872, contains a discharge curve of St. Marys Rapids similar to the others. There are many other curves in these reports, and a careful study of them will soon convince any one that errors of 2% are common in current-meter work of the best quality.

In the case of tests of the meters with a quantity of blueing injected into the water and timed over the distance between the two meters, it is gratifying to note the close agreement between the actual velocity and that indicated by the meters. The fact that the ball of blueing remained intact, however, shows that there was little turbulence in the water and, therefore, little cause for errors.

Shrinkage.—As the question of shrinkage will be of importance in many investigations, it will be of advantage to systematize the methods for computing the values of the shrinkage coefficient, and the following remarks will be of interest.

By Equation (63), Section 23,

$$k = \frac{C_2 - C_1 - (D_2 - D_1)}{C_2 D_1 - C_1 D_2} = \frac{d\,(c_2 - c_1) - c\,(d_2 - d_1)}{c_2 d_1 - c_1 d_2} \quad \ldots\ldots(204)$$

Therefore,

$$k = \frac{1 - \dfrac{c}{d}\dfrac{d_2 - d_1}{c_2 - c_1}}{\dfrac{d_1}{d} - \dfrac{c_1}{d}\dfrac{d_2 - d_1}{c_2 - c_1}} = \frac{d}{d_1}\left(\frac{1 - p\dfrac{\Delta d}{\Delta c}}{1 - p_1\dfrac{\Delta d}{\Delta c}}\right) \quad \ldots\ldots(205)$$

where $p = \dfrac{c}{d}$, $p_1 = \dfrac{c_1}{d_1}$, etc., and $\Delta d \div \Delta c$ is the mean rate of change of density per unit change of concentration.

If the reciprocal of this ratio, $\Delta c \div \Delta d$, which is the mean rate of change of concentration per unit change of density, be indicated by α, then it will be easy to show that

$$\left.\begin{aligned} k &= \frac{d}{d_1}\left(\frac{\alpha - p}{\alpha - p_1}\right) \equiv \frac{d}{d_2}\left(\frac{\alpha - p}{\alpha - p_2}\right) \\ &= \frac{d}{d_1}\left(1 - \frac{p - p_1}{\alpha - p_1}\right) \equiv \frac{d}{d_2}\left(1 - \frac{p - p_2}{\alpha - p_2}\right) \end{aligned}\right\} \quad (206)$$

It will be noticed that α is the mean slope, for a given temperature, of a curve on Plate XXXIII, Section 4, with reference to the vertical axis of co-ordinates, for the two points which indicate, respectively, the condition of the solution which is to be mixed with the salt solution

and the condition of the resulting mixture, the symbols p and d, of course, relating to properties of the strong salt solution itself. The second form of the equations is for use with the slide-rule. Mr. Groat.

The formula is perfectly general, and may be used to find the value of k'' by letting the subscripts, 1 and 2, relate, respectively, to the condition of special dilution and the condition of tail-water. Or, the subscript 1 may be changed to accord with the adopted notation, for this case, as follows:

$$k'' = \frac{d}{d'_2}\left(\frac{\alpha - p}{\alpha - p'_2}\right) = \frac{d}{d_2}\left(\frac{\alpha - p}{\alpha - p_2}\right) \quad \ldots\ldots\ldots (207)$$

Here α relates to the slope of the tangent to one of the curves of Plate XXXIII, with reference to the vertical axis of co-ordinates, at a particular point, as there is supposed to be only a very slight difference between the corresponding properties of the special dilution and tail-water at the given temperature, thus bringing the two points indicated by (d'_2, c'_2) and (d_2, c_2) into practical coincidence.

Suppose, for example, that it is desired to ascertain the value of k'' for the usual conditions of a test as given in Table 2, Section 15. The calculation by Equation (207) would then be:

$$\begin{aligned} c_2 = c'_2 &= 0.0591; \\ d_2 = d'_2 &= 0.9984; \\ p_2 = p'_2 &= 0.00006; \\ c &= 290. \\ d &= 1.1837; \\ p &= 0.245; \\ \alpha &= 1.397, \text{ as measured from Plate XXXIII.} \end{aligned}$$

The point where α is measured lies on the line for 20° temperature, as it is assumed that this was the temperature during the usual test. Therefore, by Equation (207),

$$k'' = \frac{1.1837}{0.9984}\left(1 - \frac{0.245 - 0.00006}{1.397 - 0.00006}\right) = 0.977 \ldots\ldots (208)$$

for the greater portion of the tests.

Observe that the calculation may be made with sufficient accuracy with a slide-rule, provided Equation (207) is slightly changed so as to correspond to the second form of Equations (206).

When distilled water is mixed with strong salt solution of a concentration of about 297 grammes per liter, at a temperature of 20° cent., the following results may be deduced by placing $c_1 = 0$, $p_1 = 0$, and $d_1 = 998.23$, in Equations (206).

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TABLE 94.—VALUES OF THE SHRINKAGE COEFFICIENT, k, FOR RELATIVELY SMALL RATIOS OF MIXTURE OF A SALT SOLUTION OF 296.98 GRAMMES PER LITER AND DISTILLED WATER AT A TEMPERATURE OF 20° CENT. COMPUTED FROM THE DATA OF TABLE 1 AND THE FOLLOWING ELEMENTS:

$d = 1\,187.9$, $p = 0.25$, $c_1 = 0$, $p_1 = 0$, $d_1 = 998.23$, $d \div d_1 = 1.190$, and $t = 20°$ cent. (calculation by slide-rule).

Approximate value of r	c_2	d_2	p_2	α	k	k''
0.5	197.33	1 127.6	0.175	1.525	0.995	0.999
1.0	148.08	1 096.9	0.135	1.501	0.992	0.997
2.0	101.36	1 067.0	0.095	1.474	0.989	0.994
3.0	73.409	1 048.7	0.070	1.455	0.986	0.992
4.0	57.079	1 037.8	0.055	1.443	0.984	0.990
5.0	51.705	1 034.1	0.050	1.440	0.984	0.988

In computing Table 94, the first of Equations (206) may be put in the following simple form, as c_1 and p_1 are nil:

$$k = \frac{d}{d_1}\left(1 - \frac{p}{\alpha}\right) \quad \ldots\ldots\ldots\

etc., f'' and F'' being, respectively, the ratio of mixture and the ratio of dilution necessary to convert special dilution to the condition of tail-water, in accordance with the definitions and meaning of Equations (14), Section 11, which implies that the special dilution is weaker than the tail-water, or $r' > r$.

But, by Equations (38) and analogy,

$$\left.\begin{aligned} F' = R' \frac{d_2}{d}, f' = r' \frac{d'_2}{d} \\ F = R \frac{d_2}{d}, f = r \frac{d_1}{d} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (213)$$

etc.

Therefore, by proper substitutions, Equations (211) may be transformed so as to give

$$\left.\begin{aligned} r = \frac{r'}{1 + \frac{R'}{r'}} \\ (r' > r) \\ r' = \frac{r}{1 - \frac{R}{R'}} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (214)$$

thus deriving the relations discussed in Section 35.

It is also possible to deduce these same relations by Equations (78), (90), and (91). Thus, for example,

$$r = R' r'' + r' \qquad (r > r')$$

$$r = R' r''$$

Eliminating r'' and solving for r, we have

$$r = r \frac{R'}{R'} + r', \text{ or, } r = \frac{r'}{1 - \frac{R'}{R'}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (215)$$

as before. The remaining relations may be treated in similar manner.

Conclusion.—In closing this discussion, there are a few thoughts which occur to the writer as being worthy of mention.

During the oral discussion, Mr. Wiggin asked for the cost of the tests, with the evident idea that this knowledge would be of value in determining the probable cost of future chemical tests. Unfortunately, the scope of the investigation was so much larger than would ordinarily be undertaken in turbine testing, that a mere statement of total cost without detailed analysis will be of little service for this purpose. The writer would gladly make such a detailed analysis of the cost of the complete series of studies if he thought any useful facts would be

Mr. Groat. brought to light thereby. He feels, however, that this is not the way to arrive at costs. It will be far more satisfactory for the engineer to determine his general method from a study of papers on the subject, afterward arranging the details to suit his own ideas and in conformity with his calculations. His equipment and method for the conduct of the tests thus becoming definite, it will be an easy matter to make accurate estimates of cost. It may be of interest to state that a commercial grade of salt suitable for chemical tests will cost from $1.50 per ton up (depending on the condition of the market), delivered f. o. b. cars, at the salt works.

There are two possible sources of delay and contingent expenses that may exist in connection with turbine tests in place, which it may be well to mention. Where the purchaser and maker each have the right to undertake investigations of their own during the tests, there are likely to be interferences with each other, which are sure to cause more or less delay and repetition, especially if each has a particular line of investigation differing from that of the other; and, where the tests must be conducted without interfering in any measure with the power-house output, methods become restricted, and this leads to an increase in cost.

With the preceding statements in mind, there is no objection to saying that the total cost of experiments and studies involved in the writer's paper was about $18 000. This item includes: The preliminary study; rating current-meters; numerous railroad fares; chemical testing equipment, including tanks, pumps, pipe lines, and laboratory; complete set of precision direct-current electrical testing equipment; complete set of precision alternating-current electrical testing equipment; erecting and adjusting equipment and instruments; supplies and operating expenses in the conduct of the tests; and computations. It does not include the cost of preparing the paper, except in so far as many of the tables are necessarily copies of those derived from the final computations. The writer estimates, roughly, that the value of the instruments and equipment on inventory at the conclusion of the work was $3 500, leaving a net cost of approximately $14 500.

In Section 79, attention is directed to the impropriety of adding the tail-race velocity-head to the elevation of the tail-water before determining the head acting on the turbine. By so doing, the efficiency is made to appear higher than it really is. It is not uncommon to find water issuing from draft-tubes with velocities as high as 15 or 16 ft. per sec. This is equivalent to a head of about 4 ft. If the total head on the turbine was 50 ft., this would cause the efficiency to be overestimated by nearly 9 per cent. If the tail-race velocity were 8 ft. per sec., the head would be reduced by 1 ft. If the head were only 20 ft. in this case, the efficiency would be more than 5% too high. In

making contracts, great care should be exercised not to be misled by incorporating this method of introducing an erroneous head, thereby making the computed efficiency too high. Mr. Groat.

It may be of interest to remark that titrations of salt solution were used in another investigation during the progress of the regular turbine tests. They were used to determine the location of a leak which passed from the bottom of a pond under a dam and into the tail-race of the power-house. The small stream issuing from under the dam could be sampled by simply dipping up the water in a sample container. The strong salt solution was discharged down a pipe from a barrel on a scow which was moved about over the surface of the pond. The pipe was held close to the bottom, and when it was finally placed near the intake of the leak, the titrations of samples taken down stream from the dam quickly indicated the increase of salt in the water.

In another case the location of a leak above a dam was known, and the object of the investigation was to find by means of titrations the subaqueous outlet in the bed of the river below the dam. Thus far, the writer has not received advices of the results.

It may be of value to mention that some time could have been saved in the computations had the calibration of the iron pipette, or tank calibrator, been made in the manner desired by the writer. A large vessel like this, with a contracted pipe or glass gauge at the top, can be calibrated by a scale of temperatures on the gauge so that it can be filled to contain a fixed volume whatever the temperature. This can be accomplished by graduating the scale of temperatures so that each division mark is at the proper elevation for the water surface at that temperature to correspond to the given volume. It is evident, then, for a metal vessel, that the temperature scale will begin at the top and run downward with increasing temperature. Through some misunderstanding, the temperature scale ran upward, the operation, therefore, resulted in a colossal thermometer instead of a convenient calibrator.

In the rules for the chemical laboratory, Section 88, it was said that the proper rate of draft from a burette was about 2 drops per sec. This is scarcely true for all burettes, as the sizes of the tips vary, and this changes the size of the drops. A better rule would state that the proper rate for the draft from an ordinary 75 or 100-c.c. burette is 8 or 10 c.c. per min.

Salt attracts moisture, and is likely to cake and harden when dried. One or two rough drying tests indicated that the low grade of commercial salt used in the tests carried about 6% of moisture. In other cases, a different result might be found.

The color intensity method of chemi-hydrometry should be distinguished from the method of determining mean velocity by observing

Mr. Groat. the time required for color to pass from one point in a system of conduits to another.

The temperature of samples is a very important matter in conductivity tests. However, if the method of special dilutions, or balanced samples, be adopted, the temperature correction will be eliminated.

In constructing the entrance to the outlet from the salt-solution tank, ample sectional area should be provided, else the full capacity of the tank may not be available, owing to the formation of vortices by reason of the high velocities generated. The outlet from the salt-solution tank in the present case was made of about 18 in. of 8-in. pipe, discharging vertically from the flat bottom of the tank into the 3-in. salt-solution supply pipe. In this manner, the water level in the tank can be drawn to the bottom without creating vortices or admitting air to the supply line.

For a number of years the writer has entertained the idea that a wire carrying an electric current could be used to measure the velocity of water. The current of electricity would tend to heat the wire, and the flow of water along and normal to the wire would tend to cool it, or to determine the temperature. The higher the velocity of the water, the lower the temperature of the wire. Therefore, the conductivity of the wire, or its temperature, would be a function of the velocity and, by having a calibrated apparatus for measuring the conductivity, or temperature, the velocity of the water could be determined. It does not appear, however, that much idea of direction of flow could be gained without getting into a very elaborate device. However, it may be well to mention the principle.

Referring again to the cost of turbine tests: This is a purely relative matter. Frequently, tests result in discoveries leading to improvements which pay for the tests many times over. In one case such improvements led to the recovery of a large amount of power at a cost of less than $1 per horse-power increase in capacity. The actual value of the gain was not less than $500 000, and the cost of the tests was only 1% of this value, an insignificant matter altogether. In another case, tests indicated simple changes in draft-chests, which effected a substantial increase in efficiency and power.

Attention has already been directed to the diagram of turbine performance, Plate LI. This diagram, it may be explained, gives all the information concerning turbine performance. It may be utilized to compile many kinds of tables which will be useful to designing and operating engineers and power-house superintendents. Such diagrams will be of great value in designing water-wheels to fit electric generators, and the converse. They will also be of great

value in making contracts and specifications for turbines, generators, and complete hydro-electric units. Mr. Groat.

The writer desires to express his appreciation and thanks to Messrs. Eimer and Amend for the chemical equipment used in illustration of the methods of chemi-hydrometry at the time of reading the paper of which this discussion is the conclusion. He also extends thanks to Mr. Hugh N. Davis, who very kindly assisted with the chemical operations on the same occasion.

The reader may have observed that Table 1 shows evidence of small systematic errors in the data from which it was compiled, especially for the weak concentrations. It may be well to bear this in mind when shrinkage becomes important, as a revision of Table 3 and Fig. 1 will doubtless be required when more correct data become available. This may also modify, or possibly negate, the thesis of Section 24, which states that a molecule of salt always displaces a portion of the water in which it is dissolved, however dilute the solution may be.

It may not be amiss to repeat that the writer considers Table 85, and the corresponding table of aggregates for Tests *N, O, P, Q, R* (Table 93), as showing the possibilities of current-meter work in turbulent water when the records are corrected by composite rating curves. The errors of 1 to 2% among the four methods for the individual tests are due to the fact that meters were not, in the case at hand, compared in one and the same test. The true precision attainable is apparent when all five of a group of tests are treated as a single test.

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Paper No. 1367

ADDRESS AT THE ANNUAL CONVENTION, IN PITTSBURGH, PA., JUNE 27TH, 1916

By Clemens Herschel, President, Am. Soc. C. E.

As we approach the number in our order of exercises, which by Constitutional provision calls upon the President to deliver an Address at the Annual Convention, you no doubt feel, as I feel, that we stand in the shadow of the great loss this Society has suffered in the death of its duly elected President, Dr. Elmer Lawrence Corthell. Succeeding to his duties and responsibilities, I have endeavored to follow out any plans he may have had for the Address which he was to have delivered, but have been unable to learn that he left any definite written or oral memoranda with regard to it. "The night cometh when no man can work"; and it came to our late President, before he could communicate to others the thoughts he no doubt had with regard to this address, and had intended to incorporate in it.

It will not be foreign to his desires and aspirations, however, if I choose for the subject of our considerations, a topic that may well enlist the co-operation of us all; left to struggle with the world's work now that he is gone. Just 25 years ago, this your President was the newly elected President of the Boston Society of Civil Engineers; an engineering Society, let it be noted in passing, which was chartered in 1848, and which is thus the oldest organized engineering Society in the United States. At that time he chose for his subject: "The Advancement of the Profession of the Civil Engineer",*

* *Journal*, Assoc. of Eng. Societies, March, 1891.

and I have found it interesting to read what a younger man of the same name as myself (little else remains exactly the same between us), had to say on this subject 25 years ago. So much so, that I have ventured again to choose this subject, ever old, ever new, for your contemplation to-day.

We have had of late in the technical press and in the proceedings of engineering societies, ample evidence that the advancement of the profession of the civil engineer is a cause that still has the earnest attention of the large body of thoughtful working engineers of the country. The spirit of unrest has been abroad among them, and however much, content with one's individual lot may be ethically or religiously proper, and bringing blessings in its train, yet is unrest in behalf of one's class, and an altruistic, lawful striving to better the lot of that class, a commendable, and perhaps the only effective means of achieving that end, as experience has amply shown. Engineers feel that they occupy a prominent place in the creation of the world's material welfare, and want a recognition of this fact—a recognition that a share in the world's honors is due them, in the minds and actions of their fellow-men. Especially is their unrest and a striving for increased public recognition justifiable and prominent among engineers, when the creation of State or public works is under consideration.

Time was in this country, about 100 years ago, when it was denied that a State, or the United States, should undertake any, what were then called, "Internal Improvements"; what would now be called:—Public Works. We still suffer from that self-delusive attempt to ignore necessities that always have existed in countries reckoned civilized, or progressive; that always will exist. Wars and combats by land and sea, also in the air and underneath the sea, more than ever form mile-stones in the life of nations, as it is described in history; but the silent changes wrought by custom, and without bloody contest, in that same national life, are no less marked when viewed in a calm review of the centuries. What was natural and without material harm 100 years ago, has now become burdensome, and to thinking men, absurd.

Consider the state of society in this country in which could be passed a vote in town meeting (you may read it in the annals of a certain Connecticut Valley town), that "Cornelius the Irishman have

leave to take up land, but provided he have no voice in the town affairs"; and compare it with the state of affairs to-day in that same town, the stock derived from its original population submerged, even wholly extinguished, under the output of a "melting-pot", into which have been cast, at one time or another, specimens from all the nations of the earth; many of them of a most uncouth form and texture. Hardly less opposite is the need and propriety of public works to-day, as compared with that need and propriety in the young nation of 100 years ago, barely emerging from its colonial condition, and from a wilderness, apparently left on earth on purpose for it to subdue.

Says the poet:

"Bid the broad arch the dangerous flood contain,
The mole projected break the roaring main;
Back to his bounds their subject-sea command,
And roll obedient rivers through the land,—
These honors Peace to happy nations brings;
These are imperial arts, and worthy kings."

Public works have been deemed the glory of long-established, or developed, States, since the dawn of history. To attempt to do without them, or to rely solely upon private enterprise to procure them, is a mere delusion; or notion; impracticable; and yet we have a National, and, generally speaking, forty-eight State Governments, that make no permanent provision for their construction and up-keep, in their organic law.

The expenditure of 4% of the annual United States Budget on public works is made the foot-ball of politics, and great is the cry of victory when this expenditure has been reduced to 3½%; but no word is heard about the other 96 or 96½% of expenditure of the same budget; because public works are not recognized in the Constitutions of the United States and of the States, to the extent of providing a department of the Government, and permanent officers of the Government, to construct and care for them. A large part of the lack of recognition of the civil engineer as a member of an honorable profession, and of his merits, is, in my belief, traceable to this very state of affairs.

From time to time, the technical press contains discussions as to the anomalous position held by "Junior Engineers", and "Assistant Engineers" in the service of the United States, on public works, in

charge of the Corps of Engineers, U. S. Army; the former of whom can never rise above their basic position, and the best of whom consequently leave that kind of a service as quickly as circumstances will permit. These discussions are carried on in all sorts of tempers, and, to make my own temper clear in the matter, let me express the belief, founded on long personal observation and friendships, that there does not exist an abler, a finer body of men than this same Corps of Engineers, U. S. A. Think of its genesis. A boy must have ambition, must be a select boy, to aspire to go to West Point. Competitive examinations, in the majority of cases, cull out the best of these to go there. Four years' training and schooling, and their consequent process of selection, reduces the number who enter, to the half that is able to graduate. And of these, only the four or five at the head of the class go into the Corps of Engineers. Evidently the population of the United States cannot produce better men than these four or five, thus marked for high honors. The trouble is not there. It is because the whole system of carrying on the civil engineering works of the United States is an emergency makeshift; born of a denial of the propriety of the nation to build *any* civil engineering works, and of the *necessity* of building them, when their construction could no longer be avoided, by makeshift methods.

In one, perhaps the best, of these discussions,* that have just been referred to, is contained this closing argument:

"Will any one maintain that it [Congress] is yet prepared to commission men as Civil Engineers and give them a similar [to Army, Navy, Revenue Cutter, etc.] legal status?" And if we are to stop there, the argument is no doubt a sound one. So long as Congress is not "prepared" to commission Civil Engineers to do the civil engineering work of the United States Government, that work will have to be carried on as an emergency measure, and not as a regular function of government. But this is the day of "preparedness", and sooner or later, Congress also will have to become "prepared", in this, as in many other matters.

Says Laurence Sterne, in his "Sentimental Journey": "They order this matter better in France". And not only in France, but in every country of the globe, except in Great Britain, and in the United

* *Engineering News*, May 14th and June 11th, 1908.

States; the copyist of Great Britain, which the United States has been for so long, and in so many directions. The first named, all have a Ministry of Public Works, and, carrying on the business of this department, a corps of commissioned civil engineers. To that must the United States, and every State in the Union, come, sooner or later. Commissions of laymen to rule in these matters, appointed as an emergency measure, are an anomaly in these days of a striving for efficiency. See what says the President of the Institution of Civil Engineers, of Great Britain, James Charles Inglis, in his Annual Address for 1909. He quotes an economic authority to the effect that in the great struggle of the Twentieth Century it is in other countries the aim to leave as little as possible to chance; not to allow people to muddle through *somehow*, but to eliminate as far as possible the element of the unforeseen, while carefully training the mind to cope, in an intelligent way, with any emergency.

While the British had as a rule a violent suspicion of the expert, and a strong belief in the untrained, unpaid amateur as the bright source of wisdom, allowing the expert to advise and the amateur to decide, elsewhere there was no fear of the expert. There was indeed the possible danger of red tapeism at the hands of highly trained officials, but they would be found less than the dangers arising from the decisions of well-meaning but untrained and inexperienced amateurs;—all of which is applicable to the description of a contrast between the British and all the other European; between the United States method and a *rational* procedure in the construction and administration of public works.

Says President Inglis:

"If these principles were more acted upon in this country [Great Britain], as in time they must be, I think you will agree with me that many members of the Institution would find increased scope for their energies, and added interest in the engineering problems they have to deal with, while, without doubt, better results would, on the whole, ensue for capital."

Let me illustrate how the Commission habit in State Government operates; as philosophically viewed from a distance, in recent years. A certain State conceived the idea that it would be well to develop its principal seaport by works of the civil engineer. So a Commission of laymen was immediately created to cause this to be done. The

newspapers were replete for months as to who should be the Chairman of this Commission. There was a good salary attached. Not a word as to the Civil Engineer capable of designing the necessary works, and of wisely superintending the expenditure of millions of dollars in the future. Not one word. At length the die is cast, and the favorite of the "plum-tree" enters upon a course of the, at such times customary after-dinner speeches. After months of this sort of thing, and when the novelty of the situation has worn off, not much has been heard in the daily press as to the work to be done. An engineer has indeed been somehow appointed, and certain work planned and done. Then the former after-dinner speaker resigns, or is crowded out,—it is immaterial which,—changes of political government affect and *debase* the at-no-time illustrious commission and engineering corps, and the last end of that Commission is worse than its inception;—it is *itself* swallowed up and abolished in a reshuffling of the Commissions of the State generally; and all this in the space of barely 5 years; with the simultaneous hazard of millions of money to be expended, at once and in maintenance and repairs, or reconstruction in the future, in a foot-ball game of politics.

Is it not true, does one not see, in contemplating such bits of contemporaneous history, that especially in a Republic, are commissioned civil engineers needed for the proper carrying on of the public works of the State? The reason the average Congressman, even the Executive, respects the Army Engineer, as they do, is because the former knows that the services of the latter in and to the Government will go on, when he himself will not unlikely have been wholly forgotten. He would equally respect the commissioned civil engineer.

The layman Chief-of-Bureau's tendency to lack of consideration for his expert subordinate must be checked by the expert's permanence in the service, or else we can get no results in public works such as the State ought to have.

To illustrate again, where else than in the United States would it have been possible for a dreamer of patriotic visions, in the service of the Government, to have seen those visions come true under his able guidance in the space of some 15 years, and at an expenditure of scores of millions of dollars; creating empires of arable land, where before was the desert; and after the threat of political removal

by one brief layman incumbent of the chair at the head of his bureau, finally be made to succumb, without any justification that the public ever heard of, by another such brief incumbent, to the already manifested harm of the service, and of its future.

No amateur ruling the immediate present, of an important government work and agency, should have such subversive powers of harm for their future. No matter even that he act from motives which to him at the moment seem wholly good. He may be mistaken. Certain it is that a system that enables a mere passing temporarily exalted incumbent of an office thus radically to disorganize, out of his own mere motion, a work of national usefulness and importance, is not for the good of the body politic.

In a republic, with its short-lived, kaleidoscopic, ever-changing individual leadership form of government, even more than in a conservative monarchy, the servants of the State on public works, should have fixity of tenure of office. Verily, "they order this matter better in France"; a republic; and both better, and *by the same means* better, in many a country, besides France.

More could be said in the same line of argument and illustration, but the President's Address should not be tiresome, though the advancement of the Profession of the Civil Engineer can be viewed from many another angle. Indeed, in contemplating it, one is reminded of the lament of the New England conscience,—no one felt it more than he whom inadequately I am trying to replace to you,—"so much to do; so little done".

We have, for instance, the work of combining the efforts of the various branches of the Engineering Profession, in matters of their common interest. According to Section 2 of Article II of our Constitution, there are no engineers, except civil and military engineers; the original class distinction. But so various are to-day the occasions when it is necessary to apply "the art of directing the great sources of power in Nature for the use and convenience of man", that it long ago became impracticable to have all engineers in the United States enrolled in one American Society of Civil Engineers. Instead, we have the four prominent National Engineering Societies, and many amply useful, though less prominent, such engineering Societies. And yet the number of their aggregate membership is small, compared with

the population which they serve; a prominent reason why their expressed desires are too often but as a "voice crying in the wilderness". To remedy this condition which confronts us, many have labored, in season, and out of season, to bring about joint action of the engineers of the United States in matters of a common interest. This Society has said that it believes in it, and as a step in that direction, as facilitating that sort of work, as a landmark and a beacon light to the public, that engineers are at their posts of duty and should be recognized and reckoned with, we have said that the building of the four Founder Engineering Societies on 39th Street in the City of New York shall represent to the public, the home of a power in the land. The step we have taken and which has been voted in the affirmative, 2 500 for, 390 against, is of course but a beginning. It has been worked for in a duly humble spirit, and on the principle of doing one thing at a time, but the beneficent end of that work no man can see. "Be of good comfort, Master Ridley," Latimer cried at the crackling of the flames, "play the man! We shall this day light such a candle, by God's grace, as I trust shall never be put out." Not only not put out, but increasing through the years, in effulgence, glory to the profession, and in usefulness.

I have said that the advancement of the Profession of the Civil Engineer in the United States, could be viewed from many another angle. This has nigh exhaustively been done by one of our distinguished fellow-members in a paper read February 29th, 1916, in this very city.*

There is the education of the engineer properly to be attended to. Is he, while thus in embryo, doing 24 hours of work and refreshment, in the 24 hours of the day? Is the work he is doing of the right kind?

We have many able fellow-members who are giving the travail of their lives to answer these questions.

There is reform within the active profession to be attended to. Is the typewriter machine and printing press already working overtime; running away with the drafting table and with service on the work and in the field, and with learning from Nature by experiment? What constitutes a legitimate consulting engineer? Not in the eyes of the great discerning public, but rather in those of his professional brethren.

* To be, or already published, by the Engineers' Society of Western Pennsylvania: a paper by J. A. L. Waddell, M. Am. Soc. C. E., of Kansas City, Mo.

Legislation in forty-eight States of the Union and in the Congress of the United States must be taken note of, to prevent harmful enactments, and to support those we know to be good for the body politic, no less than for the Profession of the Civil Engineer.

Not as a lamentation or a sigh of weariness, but as a cry and a call to additional, and to continued service to our profession and to mankind, let us say: As yet, *"So little done"*.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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Paper No. 1368

COHESION IN EARTH: THE NEED FOR COMPREHENSIVE EXPERIMENTATION TO DETERMINE THE COEFFICIENTS OF COHESION*

By William Cain, M. Am. Soc. C. E.

With Discussion by Messrs. Clifford Richardson, F. W. Green, E. P. Goodrich, and William Cain.

Synopsis.

The ordinary theory of earth pressure pertains to a granular material, such as clean sand, gravel, or rip-rap, supposed to be endowed with friction only. For ordinary earth, and particularly with clay, experiments given show that, in addition to friction, the earth is endowed with cohesion, and the practical importance of determining experimentally the simultaneous coefficients of friction and cohesion for every variety of earth is earnestly urged.

The results of the few experiments made on various earths and clays are given, and attention is called to the small values of the coefficients of friction and to the large values of the coefficients of cohesion in the case of consolidated earth and clay.

Harking back to the laws of friction and cohesion, first formulated by Coulomb, a simple form of apparatus for experimenting is given to illustrate principles, though it is realized that, when the earth is subjected to great pressures, a more complete testing machine is

* Presented at the meeting of February 2d, 1916.

desirable. In fact, a standard machine to which engineers could send samples of earth to be tested, seems desirable.

The results of the more recent experiments made, in both France and England, are, in some respects, so unexpected and significant that a mere glance at the figures will show the immediate need of a comprehensive system of experimenting, not only to determine the true laws of friction and cohesion of earth, but likewise to determine the coefficients of friction and cohesion for ordinary earths and clay, in various stages of consolidation.

Coulomb, about 1780, was the first to formulate the laws of friction and cohesion, as affecting a mass of earth. For a homogeneous earth, these laws may be stated thus:

(1) The maximum frictional resistance that can be exerted along any portion of a plane in the interior of a mass of earth equals the normal pressure, P_n, on the portion of the plane considered, multiplied by f, the coefficient of friction, where f is a constant for the earth considered.

(2) The maximum cohesion equals the area of the portion of the plane considered, multiplied by k, the coefficient of cohesion, or cohesion per unit of area, where k is a constant for the particular earth in question. Thus, if A denotes the area of the portion of the plane under compression and Q the total resistance to sliding along this interior plane, then,

$$Q = P_n f + k A \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

A great number of experiments have been made to determine f, the coefficient of friction for various materials, on the supposition that k was either zero or negligible; but very few have been made to determine, at the same time, the coefficients of cohesion.

A limited number of experiments, with this object in view, have been made by Collin (1846), Leygue (1885), Jacquinot and Frontard (1910), and A. L. Bell (1914). With the exception of Collin, the experimenters determined the coefficients, f and k, from the same set of experiments. Although the apparatus used by one experimenter was not the same as that used by any other, yet, in principle, the following simple device can be supposed to represent the method of

experimenting. In Fig. 1, a thin slice of earth is supposed to be placed between two metallic plaques, which are rough on the inside. These are then firmly pressed together, but without contact, and the resistance to the relative displacement of the two plaques is then measured.

Suppose the pressure to be due to a weight, and let P_n represent the sum of this weight and that of the earth and plaque above the horizontal plane of shear, AB; also, let Q be such a horizontal force applied (through a cord) to the upper plaque in the plane, AB, that sliding of the earth above the plane, AB, over the earth below it is "impending", which means that the slightest increase in Q would cause actual sliding. The earth along the plane, AB, of the area, A, resists the force, Q, by the friction and cohesion acting to the left along AB; hence, for equilibrium, at the instant motion is impending, we have,

$$Q = f\,P_n + k\,A.$$

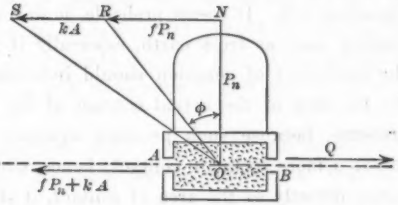

FIG. 1.

The normal reaction of the earth just below the plane, AB, $= P_n$; hence, if $ON = P_n$ is laid off vertically upward, and $NR = f\,P_n$ and $RS = k\,A$ are drawn horizontally to the left, then $NS = f\,P_n + k\,A = Q$. Also, if ϕ denotes the "angle of friction" of the earth, then $f = \tan.\,\phi$; so that the angle $NOR = \phi$. Note carefully that, in earth endowed with both friction and cohesion—coherent earth, as it may be styled—the "angle of friction" is not the surface "angle of repose." The latter is always greater than ϕ, a definite mathematical relation existing between the two angles and the depth of embankment.

If we designate by p_n the normal pressure per square foot on the plane, AB, and by q the total resistance to sliding per square foot of the plane, then, by dividing both sides of the preceding equation by A, we obtain,

$$q = f\,p_n + k = p_n \tan.\,\phi + k \ldots\ldots\ldots\ldots\ldots (2)$$

where q, p_n, and k will be expressed in tons, or pounds per square foot, according as Q and P_n are expressed in tons or pounds. As k and $\tan.\,\phi$, for any particular homogeneous earth, are constants, if the variable normal unit pressure, p_n, is laid off, from the origin, along

the axis of abscissas, then the corresponding value of q can be represented by an ordinate, and the equation is that of a straight line, making the angle, ϕ, with the axis of abscissas and cutting the axis of ordinates at the distance, k, above the origin. From the results of the experiments, simultaneous values of p_n and q can be plotted, and an average line drawn; otherwise, corresponding experimental values of p_n and q can be substituted in an equation of the foregoing type, and, from the resulting equations, the most probable values of tan. ϕ and k can be found by the method of least squares.

It may be said that it is still an open question, whether friction and cohesion, according to Laws (1) and (2), are both exerted at the same time, but the limited number of experiments which have been made seem to justify the assumptions and to verify approximately Equation (2). It seems probable, however, for very compressible substances, such as fresh earth, especially if pulverized or in lumps, that the coefficient of cohesion should increase with the normal pressure; for the area of the actual contact of the particles increases with the pressure, because such pressure squeezes the particles together and causes a more intimate contact; hence, since, by Law (2), the cohesion varies directly as the area of contact, it should prove, for such earths, the greater the larger the normal pressure.

This objection applies to a much less degree to a slice of earth cut out of a bank and experimented on in its virgin state, just as it was in the bank. However, if fresh, clayey earth is taken from a new embankment, the earth being more or less pulverized, the coefficient of cohesion is doubtless small; but if such earth is thoroughly wetted and rammed, so as to approximate to a puddle wall, its cohesion is very much increased, as the contact of the particles is more intimate than when the earth was in a friable state.

From similar considerations, it would appear reasonable to suppose that the unit cohesion in a bank of earth should increase with its depth, so that it is highly desirable to subject the earth, between the plaques, to pressures corresponding with those actually sustained in banks, say, up to 50 ft., or more, in height, in order to ascertain the variation, if any, in the coefficient of cohesion. If this variation with the height is appreciable, then an average value of the coefficient, for a particular height of bank, will have to be assumed for the imaginary

homogeneous earth to which the theory pertaining to coherent earth strictly applies.

It may be remarked, further, that Equations (1) and (2) are only valid when no relative motion of the plaques occurs. When motion once occurs, it would seem that the cohesion of the earth along AB would be destroyed, and that only friction is exerted. This is analogous to the case of the trench, in which constructors are very solicitous about placing the bracing before a break in the earth starts, for in such case, the cohesion along the surface of the earth is lost, only friction remaining, so that the pressure on the bracing is very much increased. If this reasoning is true, is not the "friction of motion" between any two bodies (wood, iron, stone, etc.), more nearly the true friction than the "friction at rest"? It is possible that cohesion as well as friction may be exerted between any two bodies at rest, so that expressions of the form of Equation (1) should be written to correspond to the results of experiments made as previously indicated. The solution of such equations will determine f, and also k, if it is not zero, as usually assumed.

The results of the experiments on earth to determine the coefficients, $f = \tan. \phi$ and k, after a method equivalent in principle to that just outlined will now be given.

In the experiments of Leygue,* the normal pressures were very small—only from 7 to 40 lb. per sq. ft.—but the laws of Coulomb were practically verified within these narrow limits. The results were:

Dry sand.....	$f = 0.70$	$\phi = 35°$	$k =$ 1.47 lb. per sq. ft.
Wet sand.....	$f = 0.85$	$\phi = 40° 22'$	$k =$ 8.28 " " " "
Very wet sand.....	$f = 1.70$	$\phi = 59° 30'$	$k =$ 6.36 " " " "
Damp fresh earth...	$f = 1.63$	$\phi = 58° 28'$	$k =$ 18.45 " " " "

Leygue states that Collin found, by an independent method, that for clayey earth, $k = 23.1$; and that for clay of little consistency, $k = 39.5$ lb. per sq. ft.

From the small value for cohesion for dry sand (only 1.47 lb. per sq. ft.), Leygue seemed to be warranted in ignoring it in analyzing the results of his carefully conducted experiments on retaining walls and boards. As the results did not agree very well with those of the

* "Nouvelle Recherche sur la Poussée des Terres," *Annales des Ponts et Chaussées*, 1885, Part II, p. 788.

sliding-wedge theory, ignoring cohesion, the writer reviewed the subject of experimental walls in an extended paper,* reaching the following results:

For walls a few feet in height, backed by dry sand, the ordinary sliding-wedge theory, ignoring cohesion, agreed in its results fairly well with those of experiments, provided the friction between the earth and wall was included from the start; but that the Rankine theory was not sustained generally by the experiments. In Leygue's experiments, however, the walls or boards were only a few inches in height, and it was found that the influence of cohesion was marked and that the results could only be harmonized with theory by including the influence of cohesion as well as that of friction. The cohesion required was very small—only about 1 lb. per sq. ft., which is a little less than that found by direct experiment—but its influence on the results was marked, owing to the small height and consequent small weight of the prism or wedge of rupture. In the course of the investigation, a complete graphical method was devised, which can be applied in ascertaining the thrusts against retaining walls and trench bracing for the coherent earth supposed. In such applications, it is absolutely necessary to find the coefficients, f and k, by the method previously detailed. In fact, the value of ϕ for consolidated earth is found to be much less than the usual so-called "angle of repose", and its low experimental value, as given in the remaining results, will doubtless give a decided shock to those not familiar with this recent experimenting.

The results of the experiments made in 1910, by MM. Jacquinot and Frontard,† on earth taken from an earthen reservoir dam which had failed owing to a considerable lowering of the water level, will next be given. The dam was constructed in the most approved manner, of the best materials, the composition of the earth tested being 60% clay, 32% silica, as an impalpable dust, and 8% silicious sand. Water was added to the earth, and it was then kneaded and compressed with the hands so as to make a firm though pasty cake, which was then inserted between the plaques. Not enough water was added to cause a lateral flow when the cake was under compression. The details of

* "Experiments on Retaining Walls and Pressures on Tunnels", *Transactions*, Am. Soc. C. E., Vol. LXXII (1911), p. 403.

† Given in some detail by Professor Résal in his work, "Poussée des Terres", Part II, "Theorie des Terres Coherentes", p. 327, Paris, 1910.

the apparatus used are not given. The results for the first series of experiments are given in Table 1.

TABLE 1.

p_n, in pounds per square foot.	$f = \tan. \phi.$ $\phi = 8°.$	k, in pounds per square foot.
692	$f = 0.14$	395
2 980	$f = 0.14$	420
5 665	$f = 0.14$	403
7 154	$f = 0.14$	448

The earth in the bank weighed 112 lb. per cu. ft., so that the recorded pressures corresponded to the vertical pressures experienced at depths of from 6 to 64 ft. for the earth in question.

After the first series of experiments had been performed, the manometers got out of order, so that a correction had to be applied, and, as there was some doubt as to the accuracy of the results, they will not be given. From all the experiments, however, some referring to the earth as it was taken out of the bank, and some to rammed earth, the quantity of water being varied, it seemed that the following conclusions, as stated essentially by Résal, could fairly be drawn:

(1) Coulomb's laws were approximately verified.

(2) $f = \tan. \phi$ ranged only from 0.14 for a soft pasty earth to 0.18 for the earth nearly dry. Neither the quantity of water used, nor the ramming or puddling, caused much variation in f.

(3) On the other hand, the quantity of water used affected the cohesion very much, and sufficient ramming could more than double the coefficient of cohesion.

The large values of k were to be anticipated, as the theory of open cuts and trenches would lead one to expect, for ordinary consolidated earth, values of k running into several hundred pounds, but the low values of ϕ (only from 8 to 10°) are somewhat startling, and indicate that our ideas with respect to the coefficient, f, and earth-pressure theory generally, may have to be considerably revised.

The values of k and ϕ, given in Table 2, lead to the same conclusions. They represent the results of experiments on clay, in its virgin state, as taken at various depths from an excavation carried

below low water, in the course of sinking a large number of monoliths which are to form the foundation of the outer sea-wall of a dockyard at Rosyth.*

The figures given represent fair average values for k and ϕ.

TABLE 2.

	k, in tons per square foot.	ϕ, in degrees.
Very soft puddle clay	0.2	0
Soft puddle clay	0.3	3
Moderately firm clay	0.5	5
Stiff clay	0.7	7
Very stiff boulder clay	1.6	16

By the term, "puddle clay", is meant a pure, homogeneous, plastic clay, free from sand or stones. As to the time element, it is stated that all the tests were of considerable duration.

In the discussion of the paper, Dr. Unwin called attention to the important fact that Collin, in 1846, found that the resistance to shear in clay produced in 12 or 15 min. was only one-third or one-fourth of that produced in 12 to 15 sec. Mr. Bell stated that experiments were made also on perfectly dry sand, for which it was found that practically $k = 0$, and that the results were in agreement with the equation, $q = f\,p_n = \tan.\ \phi\ p_n$, where, in this case, ϕ was the "angle of repose", provided the sand was rammed in the cylinder of the testing apparatus; but if it was merely poured in and shaken, then it was found that the angle, ϕ, of the equation was much less than the angle of repose.

Enough has been given to show the absolute need of a series of comprehensive tests to determine the coefficients, f and k, for every class of material. It is highly desirable, too, to have a permanent testing laboratory, with an apparatus capable of subjecting the earth to pressures varying from 0 to at least 10 tons per sq. ft., to which earth could be sent from any locality to have its coefficients, f and k, ascertained.

* The experiments were made under the direction of Mr. A. L. Bell, the results being published in *Minutes of Proceedings*, Inst. C. E., Vol. CXCIX, Session 1914–15, Part I, in a paper by Mr. Bell entitled "The Lateral Pressure and Resistance of Clay and the Supporting Power of Clay Foundations."

In designing the apparatus for testing the earth, it will be unfortunate if certain objectionable features of some previous designs are repeated. Thus, in Leygue's peculiar apparatus, a certain portion of the weight of earth and load was held up by the vertical sides of the box by friction, for which allowance had to be made. No error from this cause attaches to the very simple design shown in Fig. 1, where there is only contact of earth on earth and no contact of metal on metal. Likewise, in testing, although the effect of ramming and puddling should not be neglected, it seems reasonable to suppose that the most correct determinations of f and k could be obtained from the earth cut out of the bank or in its virgin state, especially when subjected to the pressure it bore in the bank. Then, too, careful experiments should be made to determine the effect of the time element, particularly for a substance like clay.

It seems needless to point out the practical importance of such experimenting, for it has long been recognized that, although certain theories of earth pressure give fairly correct results when the filling consists of a strictly granular material, such as sand, gravel, or rip-rap, yet when such filling consists of ordinary earth, endowed with cohesion, it is found that the theory is inadequate or, strictly speaking, that it is inapplicable, and that the theory for coherent earth must be applied.

Take, for example, the earth in the embankment or reservoir dam which failed, as previously cited. Possibly the fresh earth had a natural slope of 1 on 1½, before it was rolled, etc., and if placed behind a retaining wall, the thrust would have been computed (for $\phi = 33° 41'$) for an earth devoid of cohesion, on the supposition that the thrust thus found was in excess and, therefore, on the side of safety.

Suppose, however, that this filling was laid in horizontal layers and wetted and rolled or tamped, so that, as already found, the "angle of friction" (not the maximum inclination of the surface possible) was only 8° and k was 400 lb. per sq. ft.; then, perforce, the theory of coherent earth would have to be applied in order to obtain the true thrust. For a similar illustration, take the retaining walls designed by Gustav Lindenthal, M. Am. Soc. C. E., for a concrete viaduct 65 ft. high. These walls were thin and vertical, were placed on each side of the roadway, and were connected by tie-rods, the space between being filled with earth thoroughly tamped and consolidated.* In this

* *Engineering News*, May 6th, 1915.

case, the earth thrust can only be guessed at, unless the coefficients, f and k, are determined by experiment, in which case the theory of coherent earth will give the exact thrust. In fact, it is seen that Equation (1) exactly applies in finding the resistance to sliding along the plane of rupture.

Again, Mr. Bell's experiments, already cited, were made with the express purpose, not only of finding the earth thrust from the material surrounding the monoliths, but likewise the permissible pressures to which the clay under the monoliths could be subjected. The latter involved the theory of a possible heaving of the wall at the heel from the earth pressure behind it, or a possible heaving of the earth in front of the wall from the pressure of the latter at its toe. For the foundation, it was assumed, for the stiff boulder clay encountered, that $k = 1.5$ tons per sq. ft. and $\phi = 15$ degrees. The results would have been absurd, if k had been assumed equal to zero, as in the ordinary theory. In fact, this ordinary theory (for $k = 0$) is rarely ever applicable to foundations, which almost invariably are in consolidated earth or clay having a high coefficient of cohesion.

In addition to the subjects previously mentioned, may be cited the earth pressure on the bracing of trenches, the pressures on tunnel linings, and on piling, and the stability of slopes, none of which can be properly treated without experimental determinations of the coefficients, f and k, for the material encountered.

The theory of pressures in coherent earth is at hand—at least for homogeneous earth—but it cannot be applied without a more extended knowledge of the coefficients, f and k. In some of the recent texts, there has been a tendency to pull down existing theory. One aim of this paper is to encourage the building up of a better and more comprehensive theory—one which deals with coherent earth for the general case, and reduces to the ordinary theory for materials nearly devoid of cohesion, such as sand, gravel, or rip-rap. Such a general theory introduces all the vital elements which are necessary for a solution.

It is fully realized that changes may occur in the values of f and k from weathering, vibrations, freezing and thawing, rains, and chemical changes, so that a prudent engineer will anticipate and allow for such changes; but this, again, only emphasizes the need of experiments on a particular earth covering a long period of years.

The term, "angle of internal friction", has been introduced into engineering literature in recent years, causing much confusion and mental perplexity; for, since for dry sand, the law, $q = f\ p_n$, practically holds, then, $f = \tan.\ \phi$ is the same for the interior as for the surface, and ϕ is then truly the ordinary "angle of repose". For a homogeneous, coherent earth, however, $q = f\ p_n + k$, where k and $f = \tan.\ \phi$ are constants, though now, ϕ no longer represents the possible maximum surface inclination, but the "angle of friction", as found from experiment, after the manner previously indicated. Thus, there should be no possible confusion of terms in introducing this new (old) theory.

The subject has been brought up at this time because the Society, through one of its Special Committees, is now engaged in a general investigation of the bearing value of soils, earth pressures, etc., and, in the writer's opinion, the most hopeful method of attack is along the lines indicated herein.

DISCUSSION

Mr. Richardson.

CLIFFORD RICHARDSON,* M. AM. SOC. C. E. (by letter).—The writer has examined this paper with some interest, and requests the author to define "cohesion" in the sense that the word is used therein. Does this term denote "the state in which, or the force by which, the molecules of the same material are bound together, so as to form a continuous homogeneous mass", as defined by the Century Dictionary? or is it used to denote merely the sticking together of particles by natural attraction, without, necessarily, involving the entire homogeneity of the material, in which case the question of surfaces and films is involved? It is the writer's idea that the stability of earth and soil is more satisfactorily explained by considering them as a mixture of fine particles possessing a large area of surface to which water may or does adhere, forming a system of two phases which should be studied from the point of view of modern physical chemistry. With an increase of fineness of any material, the surface areas of these particles likewise increase to an enormous extent, and this has very great influence over its physical character.

The writer has shown† that the surface area represented by 1 gramme of sand of uniform size increases from 15 sq. cm. for a material which will pass a 10-mesh sieve up to 9 056 sq. cm. for material so fine that it does not settle out of water after agitation in less than 16 hours. The data to which reference is made are as follows:

"ONE GRAM OF SAND OF UNIFORM SIZE CONTAINS

"Mesh Sieve.	Millimeter.	Particles.	Square Centimeters.
10	1.50	212.8	15.0
20	.84	1 215.9	27.0
30	.58	3 693.6	39.4
40	.40	11 261.0	56.6
50	.26	41 005.0	87.1
80	.20	90 066.0	113.2
100	.13	328 032.0	174.2
200	.08	1 407 620.0	283.0
1 minute	.05	5 648 700.0	442.4
30 minutes	.025	46 124 900.0	905.7
2 hours	.0075	6 800 990 000.0	1 201.6
16 hours	.0025	46 124 900 000.0	9 056.6"

It will be seen, therefore, that the stability of any earth fill depends on the size of the particles and the area which their surfaces present, when considered in relation to a liquid phase, water.

It will be of interest, in connection with this subject, to study the principles of the modern chemistry of colloids which relate to surfaces

* New York City.

† In his book "The Modern Asphalt Pavement", 2d ed., p. 358.

and films.* These principles have a large bearing on the subject treated in this paper. Mr. Richardson.

F. W. GREEN,† ASSOC. M. AM. SOC. C. E. (by letter).—A rational and satisfactory theory of earth pressure has never been developed. For many years, the designer has had to content himself with an adherence to the theories of Rankine, Coulomb, Weyrauch, and others, based on various assumptions necessary to simplify the mathematical treatment, or to place his reliance on the empirical rules proposed by Sir Benjamin Baker, Trautwine, and others; and then to modify the conclusions thus obtained, in any case, by making certain arbitrary concessions to his experience and judgment. A conscientious engineer deplores such a necessity, which requires him to take chances between stability on the one hand and economy on the other. The author's effort to illuminate this subject in his paper is most interesting and praiseworthy. Mr. Green.

In the Century Dictionary cohesion is defined as follows:

"* * * the state in which, or the force by which, the molecules of the same material are bound together, so as to form a continuous, homogeneous mass. This force acts sensibly at insensible distances—that is, when the particles of matter which it unites are placed in apparent contact. At insensible distances it is a much greater, at sensible distances a much smaller force than gravitation, so that it does not follow the law of variation of the latter. It unites the particles of a homogeneous body, and is thus distinguished from *adhesion*, which takes place between the molecules of different masses or substances, as between fluids and solids, and from *chemical attraction*, which unites the atoms of a molecule together. The power of cohesion in a body is estimated by the force necessary to pull its parts asunder. In general, cohesion is most powerful among the particles of solid bodies, weaker among those of liquids, and least of all, or entirely wanting, in elastic fluids, as air and gases. Hardness, softness, tenacity, elasticity, malleability, ductility, and in crystallized bodies, cleavage, are to be considered properties dependent upon cohesion. The most powerful influence which tends to diminish cohesion is heat, as shown in the change of a solid to a liquid, or of a liquid to a gas, which is effected by it."

In all cases in actual practice, it may be said with confidence that the material to be held by a retaining wall is not homogeneous in the scientific sense. It may be all earth, but it is very doubtful, indeed, if it will ever be strictly homogeneous earth; and, if not homogeneous, it cannot be acted on by the force of cohesion. In the writer's opinion, therefore, the apparatus shown in Fig. 1 cannot measure cohesion. The ratio of Q to P_n is the coefficient of static friction of the material tested.

* Outlined in a most satisfactory manner in a book entitled "The Chemistry of Colloids and Some Technical Applications", by W. W. Taylor.

† Stamps, Ark.

Mr. Green. It is well known that the coefficient of friction is highly variable. In the Galton-Westinghouse tests, it was found that its value varied from 0.074 at 60 miles per hour to about 0.340 static, using cast-iron brake-shoes on steel tires. Static friction itself is far from being a constant quantity, moisture and temperature greatly affecting it. Rankine* gives for clay, from damp to wet, values for f varying between 1.00 for the former to 0.25 for the latter. In the writer's own experience, he has encountered an alluvial silt which, after thorough saturation, took a slope of 7:1, or a value for f of 0.143. May we not with propriety conclude, therefore, that the failure of so many retaining walls is chargeable to the use, by the designer, of a coefficient of friction which, in theory, is based on a dry granular material, but, in fact, on a wet, slippery material, rather than to a failure to introduce a cohesion factor into his computations? In other words, if proper consideration is given to the phenomenon of a variable adhesion, will the problem not resolve itself into the same thing as a constant coefficient of friction (which is the measure of the constant adhesion) plus a so-called cohesion factor (which, in reality, indicates the upper and lower limits between which variable adhesion fluctuates)? If the wall is designed for the worst anticipated conditions, as to nature of back-filling, drainage, frost, etc., instead of for "a dry granular mass without cohesion, etc.", it should not fail if properly constructed.

Cohesion is stated by many writers as being the force which prevents the sides of newly excavated trenches from caving in, and the material taking the natural slope. In the writer's opinion, which is admittedly iconoclastic, this is nothing more than another illustration of the variable nature of adhesion. The material before excavation is thoroughly compacted, that is, the particles are in especially intimate contact, so intimate, in fact, that the force of gravitation cannot at once unbalance the interior forces and thus produce movement. Eventually, moisture evaporates, atmospheric pressure enters, and then gravitation wins.

Mr. Goodrich. E. P. GOODRICH,† M. AM. SOC. C. E.—The speaker desires to commend the author for his plea for clearer thinking and for more careful analysis of physical phenomena connected with earth pressures, but does not believe that the point raised with reference to internal friction is of material moment, even though there may be theoretical reasons which lead to a possible division into sliding friction proper and cohesion. The author points out that unit cohesion will probably be found to increase with depth, and is, therefore, variable. In his discussion of the element of friction, he also suggests that a variation may occur. Should it be found that

* "Civil Engineering", p. 316.

† New York City.

Mr. Goodrich.

these two elements vary according to different laws, it may conduce to clarity of thought to analyze them separately; but should the laws of variation be found to be practically identical, or the amount of variation in one case or the other very slight, then there appears to be no practical reason for not using a single element to include both factors and including both in a common term. For the sake of clarity, the term "angle of internal friction" seems to the speaker to be as good as any.

It is not understood that the author objects to the internal friction theory as applied to many other materials, as it would seem impossible that such could be the case on the part of any one who has studied with care the analysis made by Professor Basquin in a paper* before the Western Society of Engineers in 1912. The fact that results of such marked concordance can be secured when computed by the theory of internal friction (ignoring cohesion), in comparison with tests, leads to the conclusion that the separation, suggested by the author, between sliding friction and cohesion is probably unnecessary. Table 4, which illustrates this concordance of results, has been computed by Professor Basquin.

TABLE 4—ANGLE OF RUPTURE.

Material.	ANGLE.	
	Observed.	Computed.
Cast iron, Specimen *A*	54.8°	55.3°
" " " *B*	55	53.4
Limestone	62.2	61.7
Asphalt paving brick	59.7	58.6
Milwaukee brick	58.2	58.5

The diagrams in Professor Basquin's paper, with reference to the principal stresses developed in thick cylinders, according to the theories of internal friction, maximum principal stress, maximum principal strain, and maximum shearing stress, show such a striking concordance with fact in the case of the results computed by the theory of internal friction (ignoring cohesion), that the author's suggestion would seem to be of small weight. Finally, Professor Basquin's use of the diagram of circular stress in analyzing one of the experiments on bank sand, reported in the speaker's paper on "Lateral Earth Pressures and Related Phenomena",† discloses the relative importance of cohesion and sliding friction in the case of that earth, and with a uniformity of result which is believed to go far toward demonstrating

* "The Circular Diagram of Stress and Its Application to the Theory of Internal Friction", *Journal*, Western Soc. of Engrs., Vol. XVII, p. 815.

† *Transactions*, Am. Soc. C. E., Vol. LIII, p. 272.

Mr. Goodrich.

the fact that internal friction is the largest factor in earth pressure phenomena, and that a knowledge with regard to this will make possible a fairly accurate analysis of earth pressures.

The speaker heartily agrees with the author's suggestion that the Special Committee of the Society which is making investigations of earth pressure matters should have placed at its disposal a properly equipped laboratory, if it is to do any work of value.

It is not understood that a recommendation is made as to any special testing scheme, the special device suggested by the author having been found, more than 12 years ago, to be inadequate to secure accurate results, the speaker turning finally to his testing cylinder, because it provided results which were not approached, with regard to consistency and sensitiveness, by any other device. The results of these tests with the sliding boxes, in experiments on clay, were found to be extremely difficult of interpretation. Plastic clay is almost viscous in its action, so that motion of the boxes commenced under very small stresses and actually attained a visible velocity with increases of load, even before the one at which a proper break occurred and sliding friction was developed.

Difficulties of other kinds were also encountered in the use of the boxes, which were not susceptible of being overcome by the speaker. The sensitiveness necessary in any apparatus is illustrated by the fact that his early experiments were made with cigar boxes filled with sand, and the angle of friction was measured by tilting the boxes until one slid over the other. It was found that when the first two boxes used (which were, respectively, 3 and 5 in. deep) were reversed in position, the angle of tilt at which sliding took place differed. Such a sensitiveness to load conditions was susceptible of careful determination only with accurate testing machinery, and, in consequence, the cylinder was developed. In the early experiments with the latter device, it was found that infinitesimal displacements of the soil particles produced marked changes in internal stresses.

It was thus proved indispensable that some device should be invented which would maintain internal stress and strain conditions with practical exactness. The electrical device finally installed made this possible, and it is interesting to note that the Government, in its testing experiments, has not yet been able to improve the speaker's type of design of apparatus so as to increase its sensitiveness materially. Earth is elastic to a certain extent, and as long as the strains are within this very minute elastic limit (the existence of which experiments have demonstrated), it is probable that stress changes, which are materially different from those in the earth mass in its virgin state, do not take place. The analysis of the work done by the speaker is believed to demonstrate that the experiments with

the cylinder, when analyzed by means of Rankine's formula, will give the angles of internal friction (using the word in the sense deprecated by the author), and that a series of such tests under varying pressures, when plotted in accordance with the scheme developed by Professor Basquin, will give what the author desires, *viz.*, both sliding friction and cohesion. Mr. Goodrich.

In passing, it is interesting to note that the author, in his very interesting paper on "Stresses in Wedge-shaped Reinforced Concrete Beams",* used the very term, "internal friction", which he condemns in this paper. He should doubtless be forgiven, because each has the privilege of changing his mind, but it seems to the speaker that the expression is both appropriate and descriptive, and that it will hardly be surrendered, because of a more or less theoretical idea that the internal phenomena combine two variables. The speaker wishes to repeat most emphatically his statement of belief that it is absolutely necessary to secure a vast amount of adequate information in regard to soils of all kinds. To this end it is hoped that the Special Committee of the Society will be given every opportunity to make extensive and exhaustive experiments.

WILLIAM CAIN,† M. AM. SOC. C. E. (by letter).—Both Mr. Richardson and Mr. Green refer to the definition of cohesion as given in the Century Dictionary. This definition appeals perhaps more to some physicists than to the engineer. The writer has always had in mind the following definition, as given in the Standard Dictionary: Mr. Cain.

"Cohesion is that force by which molecules of the same kind or of the same body are held together, so that the body resists being pulled to pieces. * * *

"The distinction between *cohesion* and *adhesion*, once insisted upon strongly, is not now generally regarded as fundamental. Some physicists have limited *cohesion* to particles of the same kind, others to those of the same body. Thus the force that holds the mica to the feldspar in granite would be called *cohesion* by some, and *adhesion* by others."

As is evident from this definition and the accompanying comments, the word "body" can refer to a heterogeneous mass, as a mass of earth, or a granite rock consisting of quartz, feldspar, and mica, and it is plain that there is authority—even among physicists—for the statement that any heterogeneous mass of earth may be endowed with cohesion.

Certainly, as far as the writer knows, engineers have always held this view, evidently for the reason that they are principally concerned with the resistance to sliding of the earth along a plane, and, to a limited extent, to its tensile resistance, the word "earth" referring

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 745.
† Chapel Hill, N. C.

Mr. Cain. to clay or to any heterogeneous mixture of sand or gravel with clay or clayey matter, and perhaps humus, with more or less moisture. At one extreme, we may have nearly clean, dry sand with very little cohesion; at the other extreme, pure clay with a large cohesive resistance. In fact, clay and consolidated earth act like a solid in being able to resist both tensile and shearing forces; but they differ from a solid (as iron) in this, that their elasticity is null or imperfect, and their properties vary very greatly with the heat and moisture present, with the consequent freezing and thawing, and the chemical changes that may occur. The properties of earth are likewise affected by vibration due to heavy moving trains or vehicles, so that, in consequence of the various influences cited, the time element comes in, which thus differentiates it from a solid, such as steel when properly protected from chemical changes.

The term "cohesive resistance" of earth may properly apply either to its tensile resistance or to its resistance to sliding along a plane in the earth, dependent on the viewpoint. However, as the tensile resistance of the earth is rarely called for, the term "cohesive resistance of earth", from Coulomb's time to the present, has been generally restricted to mean the resistance to sliding as affected by cohesion, exactly as given by Coulomb's two laws and the resulting Equations (1) and (2), referring to Fig. 1.

As previously stated, these two laws are still on trial, but, as far as experiments go, they are practically sustained. Thus, the results of the first series of experiments of MM. Jacquinot and Frontard, as given by the readings, when reduced to English weights and measures, are as follows:

Normal load, p_n, in pounds per square foot.	Tangential force causing rupture, q, in pounds per square foot.
692	492
2 980	837
5 665	1 196
7 154	1 450

Here p_n and q, the observed quantities of the experiments, have the meanings previously given.

On plotting the values of p_n as abscissas, and the corresponding values of q as ordinates, as shown in Fig. 2, it is found that the straight line,

$$q = p_n \text{ tan. } 8° 15' + 385,$$

very nearly passes through the plotted points. On comparing with the equation,

$$q = p_n \text{ tan. } \phi + k,$$

it is seen that $\phi = 8° 15'$, and $k = 385$ lb. per sq. ft. (cohesion). Mr. Résal, in Table 1, assumes $\phi = 8°$, and computes the corresponding values of k from the last formula, for the simultaneous values of p_n and q observed. The method just outlined seems to the writer to be more satisfactory in its results. In this first series of experiments, the cakes or pats of earth experimented on were very moist, but the apparatus was in order and the results were considered reliable. Cain.

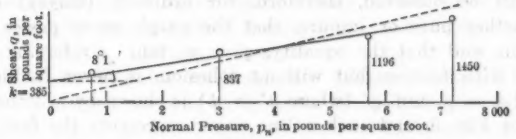

FIG. 2.

In the next series of experiments, the manometers were out of order, and a large correction had to be applied; hence the results are open to doubt, and may not be reliable. The cakes were taken directly from the earthen dam, and they contained "the minimum quantity of water compatible with plasticity". The results are as follows:

Normal load, p_n, in pounds per square foot.	Tangential force causing rupture, q, in pounds per square foot.
3 057	1 036
5 479	1 356
7 740	1 941
9 921	2 288

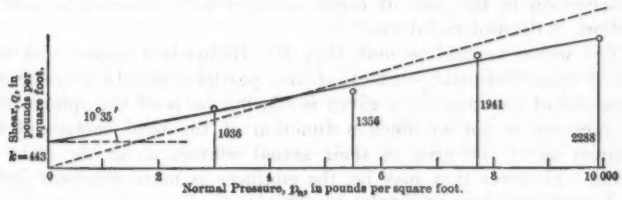

FIG. 3.

The simultaneous values of p_n and q are shown by Fig. 3, and it is found that the straight line that nearly fits the experimental results has the equation,

$$q = p_n \tan. 10° 35' + 443,$$

whence $\phi = 10° 35'$, and $k = 443$ lb. per sq. ft. On plotting to a large scale, the value of k (the intercept of the straight line on the axis of q) and the inclination of the line can be found with all desirable accuracy.

Referring now to Mr. Bell's experiments, of which only a brief summary has been given, it was found by Mr. Bell, in all the experi-

Mr. Cain.

ments on eleven different classes of soils, mainly of clay, that a straight line fitted the plotted results very well in every instance, and, further, that in no case, except for perfectly dry sand, did the straight line pass through the origin; hence the graphs, Figs. 2 and 3, are typical of the results of the experiments thus far performed, and the two laws of Coulomb, with the resulting Equations (1) and (2), seem to be verified.

It will be observed, therefore, for ordinary (clayey) earth or clay, whether pure or impure, that the graph never passes through the origin, and that the equality, $q = p_n \tan. \phi$, referring to earth endowed with friction but without cohesion, is never fulfilled. The graph of $q = p_n \tan. \phi'$ (where $\phi' > \phi$) is shown by a dotted line in Fig. 2, or Fig. 3, and such a line never represents the facts for the materials in question. The attempt to use such a line as a sort of average for the application of the ordinary theory of non-coherent earth would prove futile, because, if x is the abscissa of the intersection of the two lines,

$$q = p_n \tan. \phi' \text{ and } q = p_n \tan. \phi + k,$$

it is seen that when $p_n < x$, the value of q given by the first equation is too small and when $p_n > x$, it is too large. Thus, along a plane of rupture, the resistance to sliding would not be given correctly by the first equation for any value of p_n other than x. From all that precedes, it is evident that the statement of Mr. Green, that, in Fig. 1, "the ratio of Q to P_n is the coefficient of static friction of the material tested" is untenable, and that any attempt to apply such a conception to the case of earth endowed with cohesion as well as friction, is doomed to failure.*

The writer is glad to note that Mr. Richardson agrees with him that a more intimate contact of the particles should increase the coefficient of cohesion for a given earth, but he is of the opinion that the increase is not so much a function of the total surface of the particles as of the area of their actual contact along the plane of sliding. However that may be, the engineer is more especially interested in the experimental values.

At first sight, from examination of Figs. 2 and 3, the assumption does not seem to be realized, but it is possible that the results have not been interpreted aright. In fact, Résal suggests, in connection with the results pertaining to Fig. 3, that possibly, if ϕ was assumed equal to 8° 32′ (tan. $\phi = 0.15$) and the successive values of k are computed from the equation, $k = q - p_n \tan. \phi$, giving 578, 533, 780, 799 lb. per sq. ft., corresponding to the increasing values of p_n pre-

* The theory of coherent earth is complicated, but the writer, in his "Earth Pressure, Retaining Walls and Bins" (recently published by John Wiley and Sons), has been enabled, by aid of Mohr's "Circular diagram of stress" to develop it in the simplest and most practical manner. It is thus available whenever the coefficients, $f = \tan. \phi$ and k for the particular earth, have been ascertained by experiment.

viously given, that the (generally) increasing values of k might represent more nearly its true values. Mr. Cain.

So far as the value of q (the vital thing in earth pressure theory) is concerned, either method leads practically to the same value of this factor; so that the theory of earth pressure, which is based on the constant values of ϕ and k, as found in Fig. 3, is seen to be valid.

The supposed increase of k with the normal pressure received a striking confirmation in experimenting with the earth after it had been energetically rammed for 4 hours, when it was found that the coefficient of cohesion had about doubled.

The engineers likewise subjected the cakes or pats of earth to lateral unit pressures, two and three times as much as the normal unit pressures, the lateral pressures being exerted for an hour, then released and repeated, to consolidate the pat more thoroughly. As a result, the cohesion was found to have increased 60%, if tan. ϕ is assumed to be equal to 0.17 ($\phi = 9° 39'$).

It is a matter of great practical importance to know how to increase the coefficient of cohesion, and these experiments indicate that ramming, with use of water, will effect a material increase. With filling behind a retaining wall, well rammed and thoroughly drained, as in Mr. Lindenthal's design previously noted, the thrust of the earth should be materially decreased, and there is every prospect that the decrease may be permanent.

The writer has read with great interest the "Progress Report of the Special Committee to Codify Present Practice on the Bearing Value of Soils for Foundations",* in which the classification of soils has been done in a scientific manner, and will afford a working basis for the complete investigation of their properties, which it is to be hoped, will be carried forward, with liberal aid from the Society, to a successful conclusion. The "water content" of soils is often very loosely described, and, for definiteness, it is desirable that the tentative classification† of the Special Committee on Soils be adopted, at least temporarily.

The report of the "U. S. Bureau of Standards Sub-Committee", on Earth Experiments and Apparatus,‡ leads one to hope, from the thorough manner in which the preliminary testing of gauges, etc., is being done, that reliable practical results may be forthcoming. The wall friction, possible arching of the material in confined spaces, and even minute movements of the earth particles next the diaphragm, are bugbears to the experimenter; all of which are being investigated carefully by the Sub-Committee.

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* *Proceedings*, Am. Soc. C. E., for March, 1916, p. 343.
† *Ibid.*, p. 347.
‡ *Ibid.*, p. 353.

Mr. Cain.

In this report, the rotating gauge, Fig. 1 (c),* particularly appealed to the writer, because it seems to offer a very promising method of determining the simultaneous values of f and k for any earth. Thus, let the circle, Fig. 4, of radius r, represent the rotating disk in place, its cup-like top being filled with earth, which, in the rotation, rubs against the earth in the cylinder above it, the surface of sliding being a horizontal plane the area of which is that of the circle $= A$, say. If the earth in the cylinder is subjected to pressure, and this is distributed uniformly and acts normally to the circular top of the rotating disk, with intensity, p_n, the total normal pressure is $p_n A$, and the total frictional resistance to the movement is $p_n Af$. By a well-known theorem, the sum of the moments of all the elementary frictional resistances about the center of the circle is precisely the same as if the total frictional resistance was concentrated at a point distant $\frac{2}{3} r$ from the center. Precisely the same reasoning shows that the sum of the moments of all the resistances due to cohesion is the same as if the total cohesion, $k A$, on the circular area, was concentrated at a point distant $\frac{2}{3} r$ from its center.

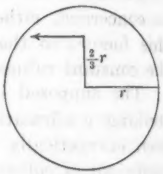

FIG. 4.

Consequently, if M is the torque or moment, due to the external force, about the center of the circle,

$$M = (p_n A f + k A) \frac{2}{3} r;$$

or,

$$q = \frac{M}{\frac{2}{3} r A} = f p_n + k = p_n \tan. \phi + k.$$

Now, this is precisely of the form of Equation (2), so that when various experimental values of q for varying pressures, p_n, have been found, they may be plotted to scale, exactly as in Fig. 2, and probable values of $f = \tan. \phi$ and k may be read off from the resulting graph.

In the foregoing investigation, the thin rim at the top of the revolving disk has been treated as if it was so much earth. If r is regarded as its inner radius, which is that of the upper surface of the earth that revolves, then the right member of the equation before the last, represents the moment of the earth resistance. To this must be added the moment of the friction of the top of the metal rim on the

* *Proceedings*, Am. Soc. C. E., for March, 1916, p. 356.

earth above, $p_n A_1 f r_1$, if A_1 is its area and r_1 is its mean radius; so that the last equation is replaced by, Mr. Cain.

$$q = \frac{M - p_n A_1 f r_1}{\frac{2}{3} r A} = p_n \tan. \phi + k,$$

which is correct for the assumptions made. This apparatus, in operation, causes a lateral pressure to be exerted on the earth corresponding to the normal pressure. It thus corresponds to the actual conditions in a mass of earth subjected to its own weight, or loaded uniformly. To a certain extent, this is true for the apparatus suggested in Fig. 1, but in this there can be no lateral pressure along the edges of the surface of rupture; so that in this respect the former apparatus is superior. It is superior, too, in giving a precise area for the surface of rupture, which cannot be said of Fig. 1, because the earth may spread beyond the confines of the boxes or plaques in which it is placed. However, the results for the plaques are not dependent on a uniform normal pressure, or unvarying coefficients, f and k, along the surface of rupture, because average values for p_n, f, and k are perhaps admissible in Equation (2); but, in the case of the rotary disk, the formula deduced is only compatible with a perfectly uniform distribution of normal pressure and for earth with constant coefficients of friction and cohesion over the whole surface of rupture. Thus, both devices have their drawbacks, and one should be used to check the other.

It is stated that Collin determined the shearing resistance of clay and then afterward, by some independent method, found its coefficient of friction. The writer is unable to see how this latter coefficient can be ascertained with any accuracy by any independent method, and he desires to emphasize the importance of determining the coefficients, f and k, from the same sets of observations as illustrated in the foregoing. In this way, and in this way only (as far as one sees), will the conditions correspond exactly to those in an actual filling behind a wall, or in a mass of earth, or a bank of any character.

In conclusion, it may be added pertinently, that the Society is extremely fortunate in having a highly trained body of scientists, like those connected with the U. S. Bureau of Standards, to undertake this exceedingly difficult subject of pressures in earth. It is needless to say that the writer is much gratified at their quick response to his call for extensive experimenting along the lines indicated in the paper.

The writer has read the discussion by Mr. Goodrich and has also reread the account of Mr. Goodrich's experimenting referred to,* and fully appreciates his efforts to advance our knowledge of the subject of

* *Transactions*, Am. Soc. C. E., Vol. LIII (1904), p. 272.

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earth pressure. He believes, however, that a satisfactory theory for ordinary earth must include the influence of cohesion, and that the discordant results found by Mr. Goodrich with different sorts of apparatus may be harmonized when cohesion and all other modifying influences are considered.

Thus, in Fig. 39 of Mr. Goodrich's paper (reproduced herein as Fig. 5), showing one box, with open top and bottom, on top of another with open top and closed bottom, both boxes being filled with earth, on top of which is a weight, it is seen that much of this weight is transferred by friction and cohesion to the sides of this upper box. If no weight is supposed, still, quite a percentage of the weight of earth in the upper box is carried by its sides. In fact, Leygue found,* for a box without a bottom, about 4 in. square in cross-section and 4 in. high, for heights of sand in the box varying from 1.2 to 3.5 in., that from 0.21 to 0.35 of the weight of sand in the box was transferred to the sides of the box. The case is analogous to that of the bin filled with sand, as to which, hundreds of experiments have shown that a part of the weight of the contained material is carried by the sides, and this proportion becomes very large for deep bins filled to the top. In such cases, the weight of earth with its load (if any) causes a lateral thrust which induces the friction on the walls, and this, with the cohesion there (if any), gives the total vertical component of the reaction of the walls. The arching effect of grain in glass bins has been observed by Mr. Ketchum.†

MODEL FOR DETERMINATION OF COEFFICIENT OF INTERNAL FRICTION.

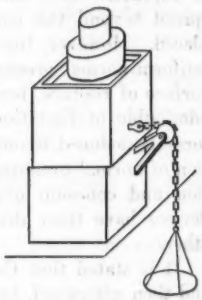

FIG. 5.

Now, recurring to the apparatus, Fig. 5, the weight transferred to the sides of the upper box, does not seem to have been considered by Mr. Goodrich, which invalidates his conclusions, so far as they refer to the experiments performed with this apparatus.

Another source of inaccuracy in any device of this kind (as in the cylinder used by Mr. Bell), where the upper box rests on the lower, is in the friction exerted between the two boxes where they are in contact, when motion is impending from the horizontal force applied. Where an appreciable proportion of the weight of earth and load is carried by the sides of the upper box, this friction may be quite appreciable, and should not be ignored, as is usually done.

* See the writer's paper on "Experiments on Retaining Walls", *Transactions*, Am. Soc. C. E., Vol. LXXII (1911), p. 414.

† Ketchum's "Walls and Bins", p. 323.

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The fact that plastic clay acts somewhat like a viscous substance, as stated by Mr. Goodrich, suggests the use of the rotating disk mentioned above to meet the difficulty. There are drawbacks in the case of the plaques, Fig. 1, the rotating disk, or the boxes, Fig. 5, which have been pointed out, and it is to be hoped that some inventor will improve upon all three.

Mr. Goodrich states, with reference to the cigar boxes (respectively 3 and 5 in. deep) filled with sand, that when the boxes "were reversed in position, the angle of tilt at which sliding took place differed". This is evidently because the weight of sand carried by the sides of the top box was different in the two cases, exactly as in the case of the Leygue experiments previously cited.

There can be no objection to the use of the term "internal friction", if it is intended to mean friction, pure and simple, as exerted in the interior of a solid or earth mass. It is used in that sense by Professor Basquin in his notable paper on the "Theory of Internal Friction", and the writer has used it in that sense in his paper on "Experiments on Retaining Walls".* The close agreement found by Professor Basquin in the observed and computed values of the angle of rupture, given in the table quoted, goes far to prove that friction and pure shear (corresponding to cohesion, as previously defined) are exerted at the same time in the materials tested. To that extent, it verifies Coulomb's two laws. In these experiments, the friction is simply proportional to the normal pressure, and the pure shear (or cohesion) varies only with the area of sliding. The total shearing force exerted, sufficient to cause rupture, equals the sum of the two resistances.

Now, unfortunately, Mr. Goodrich does not seem to entertain this view of "internal friction", when earth pressure is in question, but appears to consider it as covering sufficiently both friction and cohesion. Thus he says, where the laws of variation (of cohesion and friction) are practically indentical, "there appears to be no practical reason for not using a single element to include both factors and including both in a common term." Again, he states that the separation "between sliding friction and cohesion is probably unnecessary", and, referring to "internal friction", "that a knowledge with regard to this will make possible a fairly accurate analysis of earth pressures".

The writer feels sure that if Mr. Goodrich will study Figs. 2 and 3 and read the comments thereon, he will change his views. The theories of coherent and non-coherent earth, differ very widely in their results when the cohesion factor is large.† Thus, if the surface of the earth is horizontal, the unit horizontal thrust, q, on a vertical plane in an

* *Transactions*, Am. Soc. C. E., Vol. LXXII (1911).

† The theory of coherent earth is given in full in the writer's recent book on "Earth Pressure, Walls and Bins", from which the formula is quoted, p. 182.

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indefinite mass of earth, subjected to no external forces but its own weight, is,

$$q = p\,\frac{1 - \text{sin. } \phi}{1 + \text{sin. } \phi} - 2\,k\,\frac{\text{cos. } \phi}{1 + \text{sin. } \phi}$$

$$= \text{tan.} \left(45° - \frac{\phi}{2}\right) \left[p \text{ tan.} \left(45° - \frac{\phi}{2}\right) - 2\,k\right],$$

where p is the vertical unit pressure acting on a horizontal plane at the depth, x, considered, so that, $p = w\,x$, if w is the weight of the earth per cubic unit. The symbols, k and ϕ, have the meanings previously given. The formula does not apply when q is negative. For this upper portion of the plane, there is no pressure exerted.

This formula does not apply exactly for pressures against a rough vertical retaining wall or the vertical wall of a rough cylinder in which earth is subjected to a vertical unit pressure, p; and neither does the Rankine formula (deduced from it by making $k = 0$) apply exactly; but, suppose earth placed in Mr. Goodrich's cylinder, with the electric buzzer, etc., and that the walls are very, very smooth, so that the friction there is very small and the pressure, p, is applied. Then the foregoing formula would apply approximately; but certainly the Rankine formula does not apply when k is not zero, and any conclusions drawn from it are unwarranted. Thus, the angle of friction cannot be found by using Rankine's formula; the formula for coherent earth alone applies. On substituting several simultaneous experimental values of p and q in this formula, the values of ϕ and k can be obtained from it by trial for the earth experimented on, but the graphical method used by Professor Basquin in the paper previously referred to,* as shown in Fig. 36 of that paper, effects the solution more readily. In this figure, the ordinate at O to the line of slip, AB, measures the cohesion, as defined in this paper. For the bank sand experimented on by Mr. Goodrich, the cohesion thus found is 500 lb. per sq. ft., the angle of friction being $\phi = 32$ degrees. It is seen, from the high value of the coefficient of cohesion, that the Rankine formula is entirely inapplicable.

If the walls of the cylinder are rough, a certain amount of arching of the material is experienced, the pressure transmitted downward is not uniform, and even the formula for coherent earth does not exactly apply.

It may be inferred from the foregoing that the writer is somewhat pessimistic as to the use of gauges in small confined spaces to reach accurate results in estimating earth pressures; still, he hopes that the Special Committee on the Bearing Value of Soils will give the method a fair trial, and will succeed in overcoming the difficulties referred to. If such tests could be supplemented with experiments on retaining

* *Journal*, Western Soc. of Engrs., Vol. XVII, No. 9 (November, 1912), p. 840.

walls, or frames, 6 to 10 ft. high, backed by earth at the limit of stability, the results would be more convincing. Mr. Cain.

Electrical engineers state that there are difficulties in the way of measuring earth pressure by electrical means. If such difficulties could ever be overcome, it would afford an ideal method of finding the lateral pressure at any depth in a mass of earth or against a wall.

The writer trusts that the paper and discussions have shown fully the need for extensive experimentation to determine the coefficients of friction and cohesion in every kind of earth.

His thanks are especially due to the gentlemen who have so kindly engaged in the discussion.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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Paper No. 1369

A STUDY OF THE BEHAVIOR OF RAPID SAND FILTERS SUBJECTED TO THE HIGH-VELOCITY METHOD OF WASHING*

By Joseph W. Ellms, M. Am. Soc. C. E., and John S. Gettrust, Esq.

With Discussion by Messrs. George W. Fuller, F. H. Stephenson, E. G. Manahan, George E. Willcomb, H. Malcolm Pirnie, John H. Gregory, F. A. Barbour, George A. Johnson, James W. Armstrong, and Joseph W. Ellms and John S. Gettrust.

Synopsis.

In this paper the writers describe and discuss the development of the high-velocity method of washing rapid sand or mechanical filters, and give a record of a series of experiments obtained in washing a small experimental filter. They also describe the corrosion of a brass wire screen which, in the construction of the filter beds of the Cincinnati, Ohio, Filtration Plant, had been placed between the gravel and sand layers, and for which some other material or some modified form of construction was necessary. Additional observations are given, showing the effect of modifying the depths and sizes of the gravel layers in certain of the Cincinnati filters after removing the brass wire screen.

The paper contains tables compiled from the experimental observations, and there are a number of diagrams and curves illustrating the effect of washing the filter with wash-water at varying rates of

* Presented at the meeting of March 1st, 1916.

flow. The loss of head in the upward flow of the wash-water, produced by the strainers, varying depths and sizes of gravel, and different depths of sand, was given considerable study. The height of flotation of the sand bed during the washing process was carefully measured and recorded, and the effect of a number of factors on the height of flotation of the sand is discussed.

HISTORICAL.

In developing certain features of the design of the rapid sand filters of the Cincinnati Filtration Plant, a series of tests with an experimental filter was made between 1903 and 1905. The results obtained at that time furnished the necessary data on which the design was based. This work was carried out by one of the writers under the direction of George H. Benzenberg, Past-President, Am. Soc. C. E., and an abstract of the data secured may be found in Appendix C of Mr. Benzenberg's report as Chief Engineer of the Commissioners of Water-Works of Cincinnati, Ohio, published in 1909.

One of the principal objects of these early experiments was to determine the effect of various methods of washing the gravel and sand of the filter bed. The methods first investigated were those commonly used at that time. They consisted in forcing wash-water up through the filtering material while stirring the sand bed with mechanically rotated rakes, or by agitating with compressed air blown up through the bed before or at the time the wash-water was being applied. As a result of this preliminary work, another method of agitating the bed was developed. This consisted in forcing the wash-water up through the gravel and sand at such a rate that the sand was floated, thereby agitating and cleansing it in the same operation. This has since become known as the "high-velocity method of washing".

In these early experiments it was soon demonstrated that rotating rakes mixed the gravel and sand layers, and permitted the sand to reach the strainers and clog the openings. It was also found that compressed air blown through the bed had a disturbing influence on the two layers, and caused them to mingle to a greater or less extent. This was due in part to the type of strainers used, to the shallow depth of the gravel, and to its small size. When air and water were used together, the disturbance of the layers was much greater.

The use of wash-water at a rate of from 2.0 to 3.0 cu. ft. per sq. ft. per min. gave only slight evidence in the experimental filter that a mingling of the gravel and sand layers would result from this method of agitation. It was sufficient, however, to make it advisable to adopt precautionary measures in the design, and, with this in view, a method of forcibly holding the gravel in place by a wire-cloth screen was tried. The screen was placed between the gravel and sand layers, and was fastened rigidly. The experimental results obtained with this form of construction were so satisfactory that it was decided to place a similar screen in the large filters of the Cincinnati plant, the construction of which, at that time, was about to be commenced.

The large filter tanks of the plant were constructed with bottoms consisting of parallel hopper-shaped troughs running lengthwise of the tank, and formed by wedge-shaped concrete blocks cast in place. Spaces, 2¼ in. wide, left between the bases of adjoining rows of the parallel ridge blocks, formed the filtered water channels. These channels were covered at the top with perforated brass strainer plates. The tops of the ridges were 12 in. apart on centers, and the space from the strainer plates to these tops was filled with a graded layer of fine gravel. A brass wire-cloth with 100 meshes per sq. in. was bolted to the tops of the ridges, and rested between them on the gravel. The sand layer, having a depth of 30 in., was placed directly on the screen, as shown by Fig. 1.

The high-velocity method of washing, which had been developed experimentally, has been used with marked success during the seven years the Cincinnati Filter Plant has been in operation, and it was not until about two years after the plant was started that anything defective in the construction of the screen system was noted. It was first observed that the screen had pulled away from the bolts where it was attached to the ridges, and also at the ends of the tanks where there was lack of support for the wire-cloth and insufficient means for fastening it. Although the method of fastening the cloth was evidently mechanically defective, and although repairs were promptly made, the breaks became more frequent. It was apparent that some other factor than an inadequate method of fastening must be found to account for the constant breaking of the screen.

A careful examination of the screen showed that the brass wire was corroding, and that the short cross-wires of the cloth were broken more

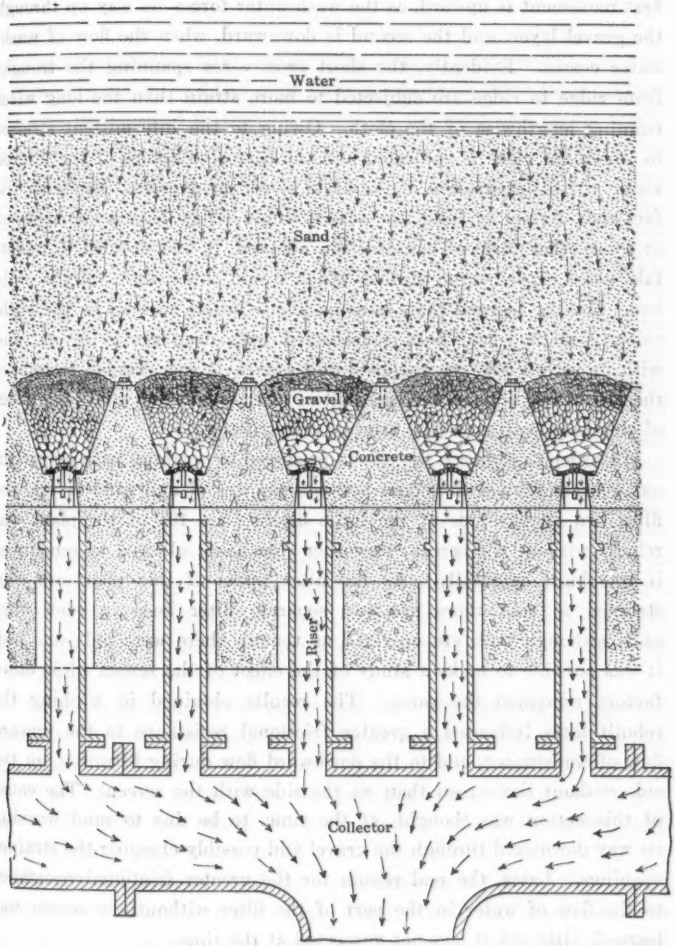


FIG. 1.

frequently than those running lengthwise. A slight movement of the screen, at the beginning and close of a wash, probably occurs. The first movement is upward, as the wash-water forces its way up through the gravel layer, and the second is downward, when the flow of wash-water ceases. Evidently, the short cross-wires spanning the troughs from ridge to ridge are subjected to more strain than the long wires running lengthwise of the cloth. Owing to the intermittent stresses to which the cloth is subjected and the corrosive action of the filtered water, the disintegration of the brass slowly progresses. Realizing the fact that, sooner or later, the screen would either have to be renewed or some other form of construction adopted, it was decided to undertake some experimental studies which would throw light on the problem. Having learned that, in some plants, filters washed by the high-velocity method had been constructed with deep gravel layers, and with no screen between the sand and gravel, it was decided to remove the screen from one of the large filters of the plant and study the effect of this modification of the original construction.

In order to compare the effects produced by washing with and without a brass wire-cloth screen between the gravel and sand layers, the filter bed on one side of the large filters (No. 19) of the plant was rebuilt without the screen, the other side being allowed to remain as it was built originally. As the filter tanks of this plant are constructed so that there are two separate filter beds in each filter unit, although both are operated as though there were only one bed, it was possible to make a study of the effect of the screen while other factors remained the same. The results obtained in washing the rebuilt filter indicated a greater frictional resistance to the upward flow of wash-water, and to the downward flow during filtration, on the side without the screen than on the side with the screen. The cause of this action was thought, at the time, to be due to sand working its way downward through the gravel and possibly clogging the strainer openings. Later, the real reason for the greater frictional resistance to the flow of water in the part of the filter without the screen was learned, although it was not suspected at the time.

On account of the difficulties encountered in working experimentally with these large filter units, in which large quantities of gravel and sand had to be handled and a great deal of time was required to make

the desired changes, it was decided to construct a small experimental filter, similar to the one experimented with in 1903.

The Experimental Filter.

The experimental filter was built of 2-in. cypress lumber. It was 3 ft. square inside, and had plate-glass windows on three sides. These windows were placed so that the action of the sand and gravel would be visible. The filter was constructed so as to be an exact vertical section of the hopper-shaped troughs and strainer-plate system used in the large 4 000 000-gal. units of the Cincinnati Filtration Plant. Holes were bored at convenient points on one side of the tank, and glass gauge tubes were attached to fittings screwed into these holes. The height to which the water rose in these tubes was the measure of the pressure, in feet of water, at the point where the gauge was attached. Fig. 2 is a section through the filter showing the principal dimensions and also the location of the various gauges. The hopper-shaped troughs which hold the gravel were of the same shape and size as those in the large filters. The strainer plates used were also the same as those in the large filters. These plates were of brass, and had four rows of $\frac{3}{32}$-in. holes punched in them, each hole being $\frac{3}{4}$ in. from adjacent holes. The combined area of the openings in the strainer plates was 0.3% of the sand area of the filter. There were two gutters for draining off the wash-water, their sides being at an angle of 45° to the side of the filter tank. These gutters in the experimental filter differed from the deep gutters with straight sides which are used in the large filter units of the plant.

All the experiments were made with the water entering and leaving the filter in a reverse direction to that followed by the water during the process of filtration, that is, all experiments were made by washing the filter. As this process lifts the sand and causes it to move up and down, it is the one which is most likely to cause the gravel and sand to mix.

In measuring the flow of the wash-water to the filter, the writers used a 3-in. Worthington turbine meter which was purchased and calibrated expressly for this purpose. The wash-water was delivered under the strainer plates at four points at the bottom of the filter. The head, in feet of water at various points in the filter during the process of washing, was measured by the eight gauge glasses pre-

viously described. Back of these gauge glasses there were scales painted white, and marked off in feet and tenths, and these were set so that the zero point of each was at the same elevation. This datum line was taken arbitrarily at the inside bottom of the filter tank, all

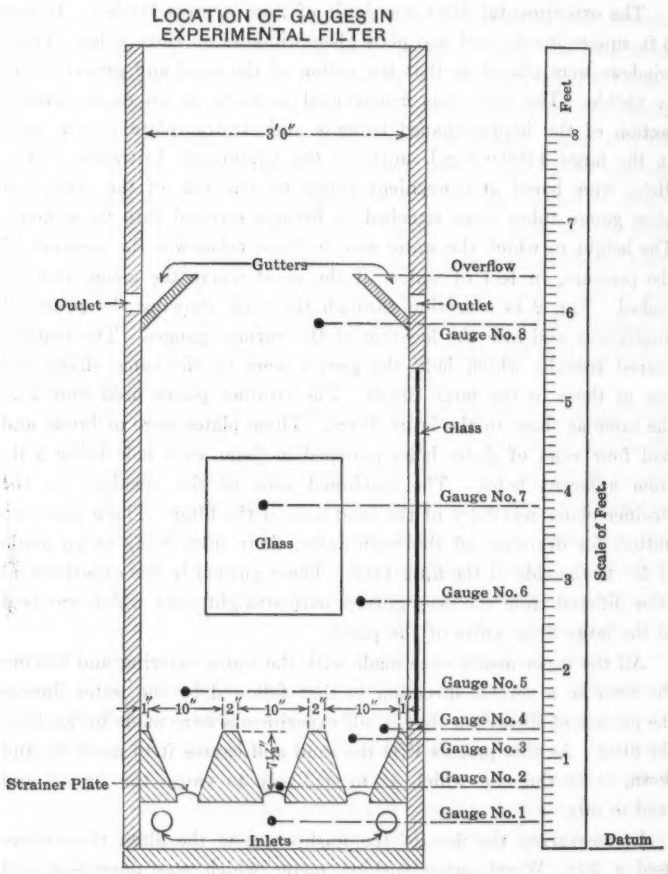


FIG. 2.

readings of the various gauges being in feet above this point. Gauge No. 1 was set under the strainer plates, at Elevation 0.3, and was approximately 0.3 ft. below them. This gauge gave a measure of the total head required to force the wash-water through the filter. Gauge

No. 2 was set immediately above the middle strainer plate, at Elevation 0.68. For a given quantity of wash-water passing through the filter, the difference in the readings between Gauges Nos. 1 and 2 gave the loss of head through the strainer plates. Gauge No. 3 was set at Elevation 1.23, so as to be just below the brass wire screen used to separate the 7½ in. of gravel from the sand. Gauge No. 4 was set at Elevation 1.32, so as to be just above the screen. Gauge No. 5 was set at Elevation 1.75, and was at the dividing line between the sand and gravel when 14 in. of gravel were used. Gauges Nos. 6, 7, and 8 were set, respectively, at Elevations 2.75, 3.82, and 5.84. Gauge No. 8 was placed so as to be above the floating sand and thus give a true measure of the elevation of the top of the water in the filter. The top of the gutters was at Elevation 6.38.

In making the tests, ordinary washed river sand was used, the mechanical analysis being given by Fig. 3. The sand had an effective size of 0.41 mm. and a uniformity coefficient of 1.41.

Conditions Under Which the Various Tests Were Conducted.

The writers state the results of seventeen tests with the experimental filter, and in Table 1 give the conditions under which these various tests were conducted.

TABLE 1.—Conditions Under Which Tests were Conducted.

Test No.	Depth of gravel, in inches.	Depth of sand, in inches.	Remarks.
1	None.	None.	Strainer plates only in place.
2	7½	None.	Strainer plates and gravel in place.
3	7½	None.	Brass wire screen placed over gravel.
4	7½	20	Brass wire screen between sand and gravel.
5	7½	25	Brass wire screen between sand and gravel.
6	7½	30	Brass wire screen between sand and gravel.
7	7½	20	No screen between sand and gravel.
8	7½	25	No screen between sand and gravel.
9	7½	30	No screen between sand and gravel.
10	14	None.	No screen.
11	14	20	No screen between sand and gravel.
12	14	25	No screen between sand and gravel.
13	14	30	No screen between sand and gravel.
14	18	None.	No screen.
15	18	20	No screen between sand and gravel.
16	18	25	No screen between sand and gravel.
17	18	30	No screen between sand and gravel.

Note.—For each of these tests, a tabulation of the data may be found on the "test sheet" having the same number as that of the test; and plotted curves of the data will be found on the "curve sheets" having numbers corresponding to those of the tests. The details of Tests Nos. 5, 8, 12, and 16, however, are omitted in the paper, as they show the conditions in washing with a depth of 25 in. of sand. The conditions of washing for 20 and 30 in. are shown in the other tests, and there are no marked characteristics for the 25-in. depth which are not indicated by the tests for the 20 and 30-in. depths.

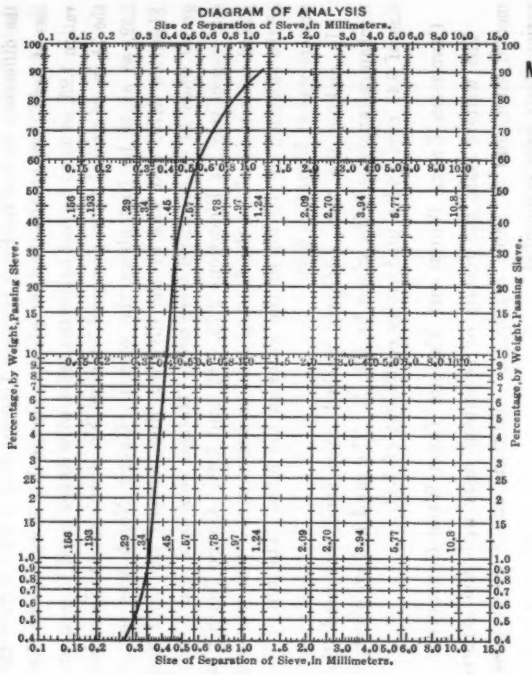


FIG. 3.

CINCINNATI, OHIO

MECHANICAL ANALYSIS OF FILTERING MATERIAL

No. of Sample EXPERIMENTAL FILTER

Location of Deposit AVERAGE (ENTIRE DEPTH)

Date of Collection

Date of Analysis

Remarks

Size of Separation of Sieve No.	Mm.	Weight Passing Sieve, in Grammes	Percentage, by Weight, Passing Sieve
2	10.8		
4	5.77		
6	3.94		
8	2.70		
10	2.09		
16	1.24		
20	0.97	85.0	85.0
24	0.78	77.0	77.0
30	0.57	59.0	59.0
40	0.45	26.0	26.0
50	0.34	1.0	1.0
60	0.29	0.5	0.5
80	0.193	0.2	0.2
100	0.156	0.0	0.0

Amount examined 100 Grammes

Effective Size 10 per cent finer than 0.41 mm.

60 per cent finer than 0.58 mm.

Uniformity Coefficient 1.41

Per cent finer than 0.20 mm. =

" " " " *0.25 mm.* =

" " " " *0.85 mm.* =

When a depth of 7½ in. of gravel was used, it was graded and placed as follows:

First layer: 1-in. layer (hand-placed) that passed a 1-in. mesh sieve and was held on a ½-in. mesh sieve;

Second layer: 6 in. of gravel that passed through a ½-in. mesh sieve and was held on a sieve with 10 meshes per lin. in.;

Third layer: ½-in. layer of gravel that passed a 10-mesh sieve and was held on a sieve with 12 meshes per lin. in.

This arrangement of the gravel was the same as in the original construction of the 4 000 000-gal. filter units.

When a depth of 14 in. of gravel was used, it was graded and placed as follows:

First layer: 2 in. of gravel (hand-placed) that was held on a 1-in. mesh screen but would pass through a 2-in. mesh screen;

Second layer: 2 in. of gravel that passed a 1-in. mesh screen and was held on a ¾-in. mesh screen;

Third layer: 3 in. of gravel that passed a ¾-in. mesh screen and was held on a ½-in. mesh screen;

Fourth layer: 3 in. of gravel that passed a ½-in. mesh screen and was held on a ¼-in. mesh screen;

Fifth layer: 4 in. of gravel that passed a ¼-in. mesh screen.

When a depth of 18 in. of gravel was used, it was graded and placed as follows:

First layer: 2 in. of gravel that was held on a 1-in. mesh screen but would pass through a 2-in. mesh screen;

Second layer: 3 in. of gravel that passed a 1-in. mesh screen and was held on a ¾-in. mesh screen;

Third layer: 3 in. of gravel that passed a ¾-in. mesh screen and was held on a ½-in. mesh screen;

Fourth layer: 4 in. of gravel that passed a ½-in. mesh screen and was held on a ¼-in. mesh screen;

Fifth layer: 6 in. of gravel that passed a ¼-in. mesh screen.

Method of Making Observations.

In making the tests, a 2-in. gate-valve, which admitted the wash-water under the strainer plates, was gradually opened enough to give sufficient upward velocity to the wash-water to cause flotation of all

the sand in the filter. Readings of the eight gauges were then recorded as shown by the height of the water in them, and readings of the water meter were taken at the start and finish of the observations made with a certain rate of flow of the wash-water. The time interval between the readings of the meter was taken with a stop-watch. The height to which the sand floated during the washing process, above that which it occupied when the filter was idle, was measured by observations through the plate-glass windows. Measurements were also made by using small metal cups, open at the top and arranged on a steel plate so that the top of the lowest cup was 2 in. from the bottom of the plate, the top of each cup being 2 in. above the top of the next lower one. The arrangement of this apparatus is shown by Fig. 4.

When starting a test, the bottom edge of the sand catcher was placed level with the surface of the sand as it lay in its normal position after being washed and hydraulically graded. The height to which the sand floated could be determined within less than 2 in. by withdrawing the cups from the filter at the close of the test and noting the height, in inches, from the bottom edge of the plate to the top of the highest cup which was filled with sand. During most of the tests, the water was clear enough to read the height of the sand without withdrawing the cups, and it was possible to interpolate readings of the height between adjacent cups.

In conducting a test, the upward velocity of the wash-water was increased step by step, readings being taken as previously stated, and was finally increased to about 2.1 ft. per min., the latter rate being as great as it was possible to obtain. The supply of wash-water was obtained from the same source as that which furnishes the wash-water for the 4 000 000-gal. units of the filter plant. These large filter units are washed with an upward velocity of the wash-water of 2 ft. per min. This rate is calculated from the area of the filter unit and the quantity of water used per minute in washing. No allowance is made for the space taken by the sand particles or by the wash-water troughs.

The wash-water used was filtered water which had been raised by centrifugal pumps to an elevated reservoir. This reservoir has an overflow, thus making it possible to operate one of the centrifugal pumps continuously and thereby maintain a constant head in the wash-water tank while making tests with the experimental filter. The

water from the overflow of the wash-water reservoir passes back through a pipe line to the suction well of the pumps. The washing of the large filter units was suspended while a test was being made with the

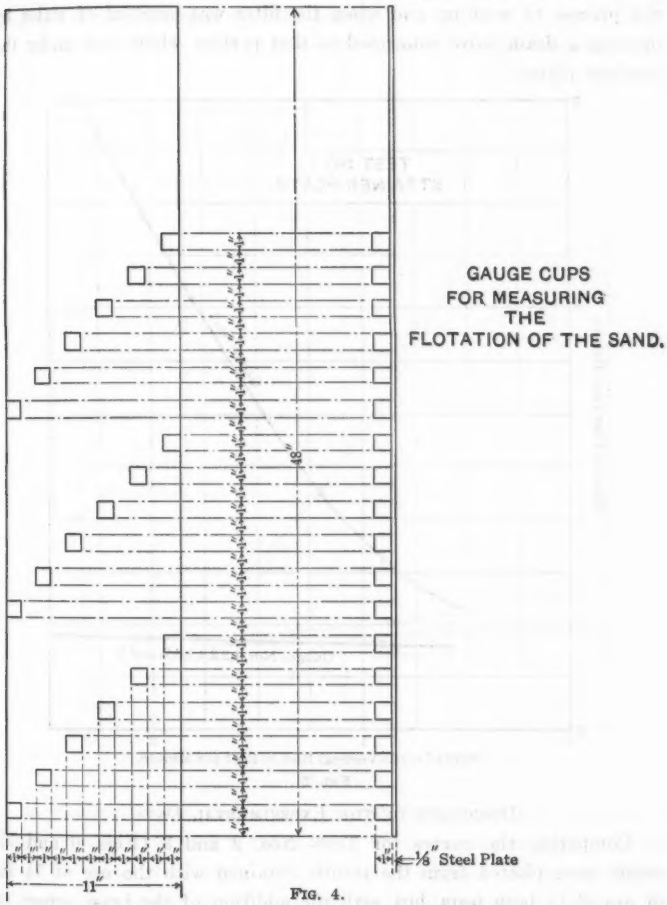


FIG. 4.

experimental filter, so that there would be no variations in the upward velocities of the wash-water in the latter on account of changes in the acting head, except those purposely made by the person conducting the test.

During all the tests the effect on the sand and gravel was observed carefully through the plate-glass windows. The behavior of the sand and gravel at the plane dividing them was noted, particularly during the process of washing and when the filter was emptied of water by opening a drain valve connected to that portion which was under the strainer plates.

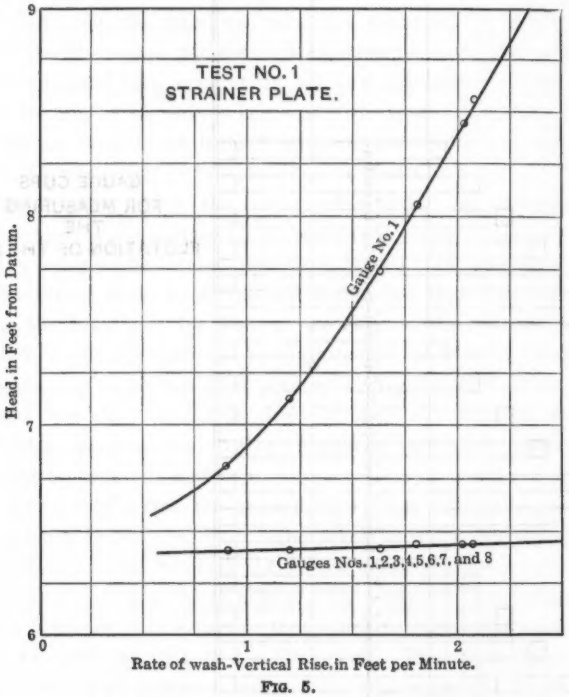


FIG. 5.

DISCUSSION OF THE EXPERIMENTAL DATA.

Comparing the curves for Tests Nos. 2 and 3 (Figs. 6 and 7), which were plotted from the results obtained with the use of 7½ in. of gravel in both tests, but with the addition of the brass screen in Test No. 3 and without it in Test No. 2, it is evident that the introduction of the screen added very little to the head necessary to force the water through the filter. The head, as recorded by Gauge No. 3, which was directly below the screen, was identical for the two tests.

Gauges Nos. 1 and 2 showed a head approximately 0.02 ft. higher in Test No. 3 made with the screen, than in Test No. 2 made without the screen. The screen used was new, and the openings between the wires had not become plugged up with gravel.

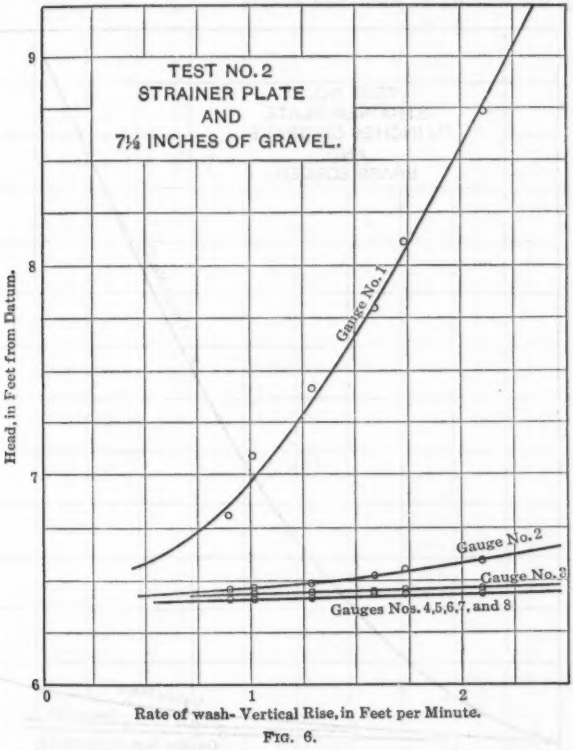


FIG. 6.

A comparison of the curves plotted from the data obtained in Tests Nos. 4, 6, 7, and 9 (Figs. 9 to 11, inclusive) in which 20 and 30 in. of sand were used, both with and without the screen, shows that, for the same depth of sand, the total head necessary to force the wash-water through the filter, at equal upward velocities, was practically the same with or without the screen. Gauges Nos. 2 and 3 showed a slightly increased loss of head in the three tests without the screen over those with the screen. The tests with the screen were con-

ducted previously to those without the screen. It is probable that, in walking over the gravel when the screen was removed, the gravel around the gauge outlets was disturbed and became lodged in such a position as to cause the upward velocity of the wash-water to increase slightly the reading of these two gauges.

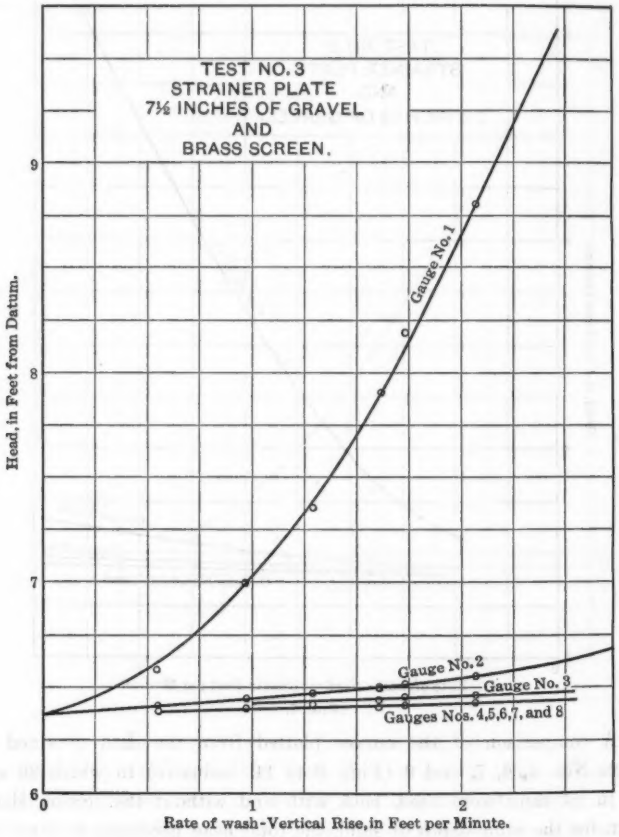


FIG. 7.

In Test No. 5 (the curves for which are not reproduced herein), with an upward velocity of the wash-water of 1.6 ft. per min., the pressure on Gauge No. 7 began to increase rapidly. At this point the

TEST No. 1.

Conditions: Strainer Plates, Only, in Place.

TEST No. 2.

Conditions: Strainer Plates, and 7½ in. of Gravel in Place.

Reading No.	Upward velocity of wash-water, in feet per minute.	Rise of sand, in inches.	Gauge Reading.								Upward velocity of wash-water, in feet per minute.	Rise of sand, in inches.	Gauge Reading.							
			No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	No. 6.	No. 7.	No. 8.			No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	No. 6.	No. 7.	No. 8.
1	0.9	.....	6.81	6.4	6.4	6.4	6.4	6.4	6.4	6.4	0.81	.....	6.81	6.45	6.41	6.40	6.40	6.40	6.40	6.40
2	1.2	.....	7.12	6.41	6.41	6.41	6.41	6.41	6.41	6.41	1.02	.....	7.07	6.47	6.43	6.41	6.41	6.41	6.41	6.41
3	1.63	.....	7.74	6.42	6.42	6.42	6.42	6.42	6.42	6.42	1.30	.....	7.41	6.49	6.44	6.42	6.42	6.42	6.42	6.42
4	1.81	.....	8.06	6.43	6.43	6.43	6.43	6.43	6.43	6.43	1.60	.....	7.79	6.52	6.45	6.43	6.43	6.43	6.43	6.43
5	2.03	.....	8.43	6.43	6.43	6.43	6.43	6.43	6.43	6.43	1.74	.....	8.12	6.55	6.46	6.43	6.43	6.43	6.43	6.43
6	2.09	.....	8.55	6.43	6.43	6.43	6.43	6.43	6.43	6.43	2.11	.....	8.73	6.58	6.47	6.43	6.43	6.43	6.43	6.43

TEST No. 3.

Conditions: Strainer Plates, 7½ in. of Gravel, and Brass Screen in Place.

TEST No. 4.

Conditions: Strainer Plates, 7½ in. of Gravel, Brass Screen, and 20 in. of Sand in Place.

1	0.56	.....	6.59	6.41	6.39	6.39	6.39	6.39	6.39	6.39	0.83	1.7	8.13	7.75	7.68	7.59	7.33	6.57	6.40	6.40
2	0.98	.....	7.0	6.45	6.41	6.41	6.41	6.41	6.41	6.41	1.08	3.0	8.44	7.77	7.74	7.65	7.38	6.62	6.41	6.41
3	1.29	.....	7.36	6.47	6.42	6.41	6.41	6.41	6.41	6.41	1.26	4.0	8.69	7.79	7.71	7.67	7.39	6.65	6.41	6.41
4	1.62	.....	7.88	6.50	6.44	6.42	6.42	6.42	6.42	6.42	1.46	4.8	9.0	7.82	7.78	7.68	7.40	6.68	6.41	6.41
5	1.73	.....	8.18	6.52	6.45	6.43	6.43	6.43	6.43	6.43	1.73	6.0	9.39	7.84	7.79	7.70	7.40	6.70	6.42	6.42
6	2.08	.....	8.80	6.56	6.46	6.43	6.43	6.43	6.43	6.43	2.04	7.5	10.0	7.9	7.83	7.72	7.42	6.74	6.43	6.43

TEST NO. 6.

Conditions: Strainer Plates, 7½ in. of Gravel, Brass Screen, and 30 in. of Sand in Place.

Reading No.	Upward velocity of wash-water, in feet per minute.	Rise of sand, in inches.	Gauge Reading.							
			No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	No. 6.	No. 7.	No. 8.
1	1.08	3	9.16	8.43	8.38	8.31	8.03	7.25	6.48	6.41
2	1.28	5	9.45	8.48	8.43	8.35	8.05	7.28	6.55	6.41
3	1.42	6	9.68	8.52	8.45	8.38	8.08	7.30	6.60	6.42
4	1.60	7	9.98	8.54	8.48	8.4	8.09	7.32	6.65	6.42
5	1.73	9	10.19	8.58	8.50	8.4	8.12	7.37	6.70	6.43
6	2.05	11	10.72	8.65	8.53	8.42	8.14	7.40	6.78	6.43

TEST NO. 7.

Conditions: Strainer Plates, 7½ in. of Gravel, and 20 in. of Sand. Without Brass Screen.

Upward velocity of wash-water, in feet per minute.	Rise of sand, in inches.	Gauge Reading.							
		No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	No. 6.	No. 7.	No. 8.
0.78	1.8	8.13	7.85	7.72	7.61	7.34	6.55	6.40	6.40
1.09	3.3	8.50	7.89	7.80	7.67	7.37	6.60	6.41	6.41
1.26	4.0	8.72	7.92	7.83	7.68	7.38	6.63	6.41	6.41
1.43	5.0	8.91	7.94	7.86	7.70	7.39	6.67	6.42	6.42
1.69	6.3	9.38	7.98	7.90	7.73	7.40	6.70	6.42	6.42
2.04	8.0	10.01	8.03	7.94	7.73	7.43	6.76	6.43	6.43

TEST NO. 9.

Conditions: Strainer Plates, 7½ in. of Gravel, and 30 in. of Sand. Without Brass Screen.

1	0.65	2	8.73	8.51	8.41	8.30	8.03	7.20	6.43	6.40
2	0.93	4	9.14	8.60	8.51	8.36	8.08	7.32	6.58	6.41
3	1.29	6	9.45	8.64	8.56	8.38	8.08	7.33	6.66	6.41
4	1.53	8	9.80	8.69	8.60	8.40	8.09	7.34	6.71	6.42
5	1.78	10	10.17	8.73	8.63	8.42	8.11	7.38	6.77	6.43
6	2.03	12	10.65	8.76	8.65	8.43	8.12	7.41	6.84	6.43

TEST NO. 10.

Conditions: Strainer Plates and 14 in. of Gravel.

0.83	.....	6.92	6.50	6.47	6.47	6.40	6.40	6.40	6.40
1.14	.....	7.31	6.54	6.51	6.50	6.41	6.41	6.41	6.41
1.39	.....	7.71	6.58	6.54	6.53	6.41	6.41	6.41	6.41
1.64	.....	8.13	6.63	6.57	6.56	6.42	6.42	6.42	6.42
1.88	.....	8.50	6.67	6.60	6.58	6.42	6.42	6.42	6.42
2.07	.....	9.00	6.73	6.64	6.61	6.43	6.43	6.43	6.43

TEST NO. 11.
Conditions: Strainer Plates, 14 in. of Gravel, and 20 in. of Sand.

Reading No.	Upward velocity of wash-water, in feet per minute.	Rise of sand, in inches.	Gauge Reading.							
			No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	No. 6.	No. 7.	No. 8.
1	0.79	1.5	8.23	7.84	7.82	7.81	7.75	6.93	6.40	6.40
2	1.03	2.8	8.58	7.92	7.89	7.88	7.76	6.97	6.41	6.41
3	1.25	4.3	8.94	7.99	7.94	7.93	7.78	7.00	6.42	6.42
4	1.44	5.3	9.29	8.04	7.99	7.98	7.79	7.03	6.45	6.42
5	1.65	6.5	9.68	8.10	8.03	8.01	7.80	7.05	6.49	6.43
6	2.08	9.0	10.51	8.19	8.10	8.07	7.82	7.09	6.56	6.43

TEST NO. 13.
Conditions: Strainer Plates, 14 in. of Gravel, and 30 in. of Sand.

Upward velocity of wash-water, in feet per minute.	Rise of sand, in inches.	Gauge Reading.							
		No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	No. 6.	No. 7.	No. 8.
0.68	2	8.80	8.51	8.50	8.49	8.40	7.63	6.78	6.40
0.95	4	9.14	8.62	8.59	8.58	8.44	7.67	6.88	6.41
1.15	6	9.44	8.69	8.66	8.65	8.46	7.70	6.93	6.41
1.36	8	9.77	8.76	8.71	8.70	8.47	7.71	6.98	6.42
1.48	9	9.97	8.79	8.74	8.72	8.48	7.71	7.00	6.42
2.02	14	11.11	8.92	8.85	8.82	8.51	7.78	7.13	6.43

TEST NO. 14.
Conditions: Strainer Plates and 18 in. of Gravel.

Reading No.	Upward velocity of wash-water, in feet per minute.	Rise of sand, in inches.	Gauge Reading.							
			No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	No. 6.	No. 7.	No. 8.
1	0.74	.....	6.92	6.50	6.47	6.47	6.45	6.39	6.40	6.40
2	1.18	.....	7.61	6.58	6.55	6.54	6.49	6.41	6.41	6.41
3	1.44	.....	8.14	6.64	6.59	6.58	6.51	6.41	6.41	6.41
4	1.70	.....	8.69	6.69	6.63	6.61	6.54	6.42	6.42	6.42
5	1.93	.....	9.08	6.73	6.66	6.65	6.56	6.42	6.42	6.42
6	2.05	.....	9.60	6.78	6.70	6.68	6.57	6.43	6.43	6.43

TEST NO. 15.
Conditions: Strainer Plates, 18 in. of Gravel, and 20 in. of Sand.

Upward velocity of wash-water, in feet per minute.	Rise of sand, in inches.	Gauge Reading.							
		No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	No. 6.	No. 7.	No. 8.
0.86	2	8.50	8.03	8.01	8.00	7.94	7.26	6.47	6.40
1.21	4	9.02	8.13	8.10	8.09	8.00	7.30	6.57	6.41
1.55	6	9.62	8.22	8.18	8.17	8.00	7.31	6.65	6.42
1.78	8	10.10	8.30	8.23	8.22	8.10	7.32	6.69	6.42
1.94	9	10.46	8.35	8.28	8.26	8.12	7.33	6.72	6.43
2.04	10	10.75	8.38	8.31	8.28	8.14	7.34	6.74	6.43

TEST NO. 17.
Conditions: Strainer Plates, 18 in. of Gravel, and 30 in. of Sand.

1	0.74	2	8.95	8.62	8.61	8.60	8.55	7.93	7.08	6.40
2	1.13	6	9.56	8.81	8.78	8.77	8.69	7.99	7.20	6.41
3	1.46	10	10.13	8.93	8.88	8.87	8.77	8.02	7.27	6.42
4	1.68	13	10.55	9.01	8.94	8.93	8.80	8.04	7.30	6.42
5	1.83	15	10.87	9.07	9.00	8.98	8.84	8.06	7.34	6.43
6	2.02	17	11.32	9.14	9.06	9.04	8.88	8.08	7.39	6.46

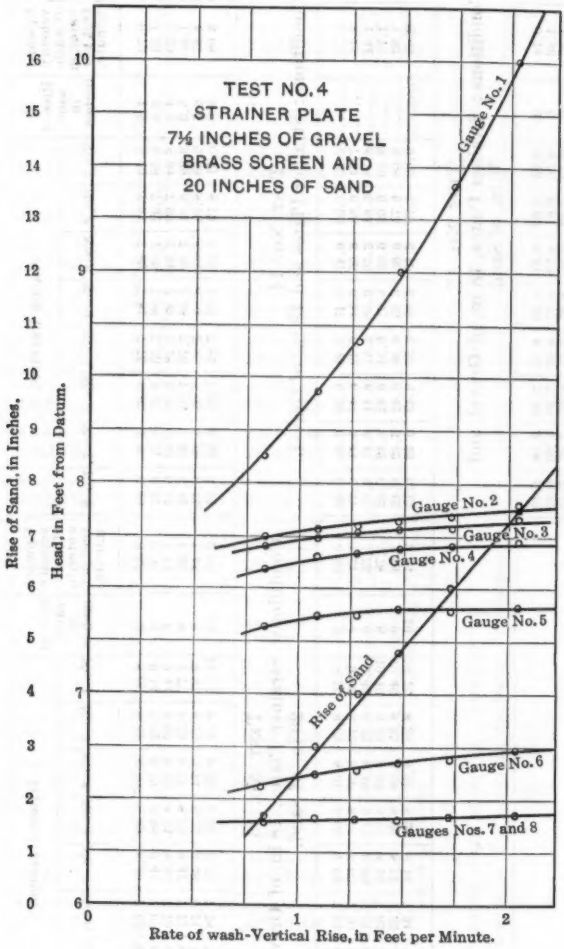

TEST NO. 4
STRAINER PLATE
7½ INCHES OF GRAVEL
BRASS SCREEN AND
20 INCHES OF SAND
Gauge No. 1
Gauge No. 2
Gauge No. 3
Gauge No. 4
Gauge No. 5
Rise of Sand
Gauge No. 6
Gauges Nos. 7 and 8
Rise of Sand, in Inches.
Head, in Feet from Datum.
Rate of wash-Vertical Rise, in Feet per Minute.

FIG. 8.

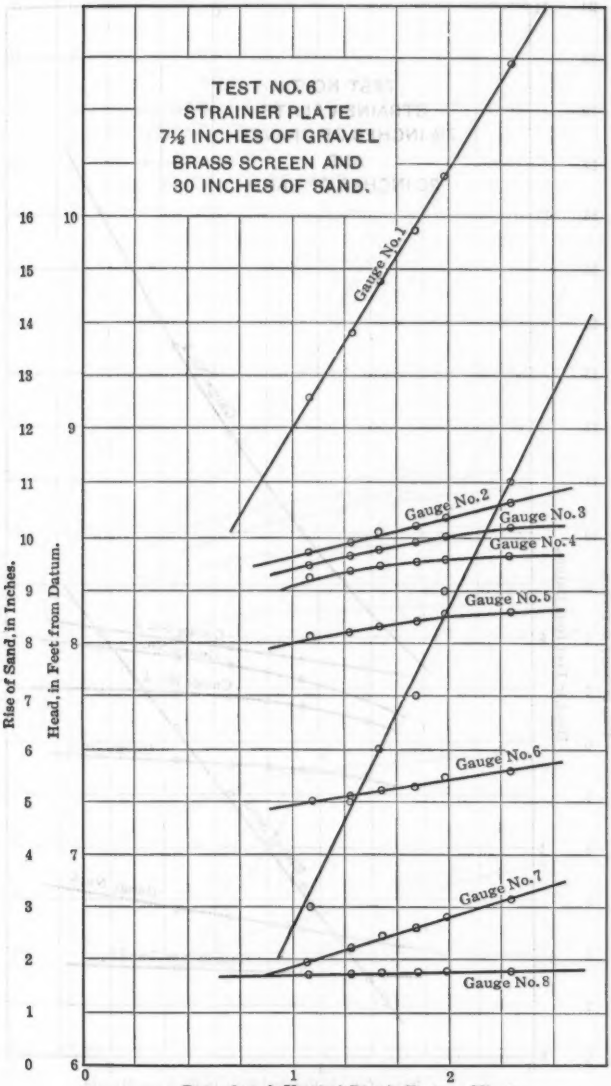

TEST NO. 6
STRAINER PLATE
7½ INCHES OF GRAVEL
BRASS SCREEN AND
30 INCHES OF SAND.
Gauge No. 1
Gauge No. 2
Gauge No. 3
Gauge No. 4
Gauge No. 5
Gauge No. 6
Gauge No. 7
Gauge No. 8
Rise of Sand, in Inches.
Head, in Feet from Datum.
Rate of wash-Vertical Rise, in Feet per Minute.

FIG. 9.

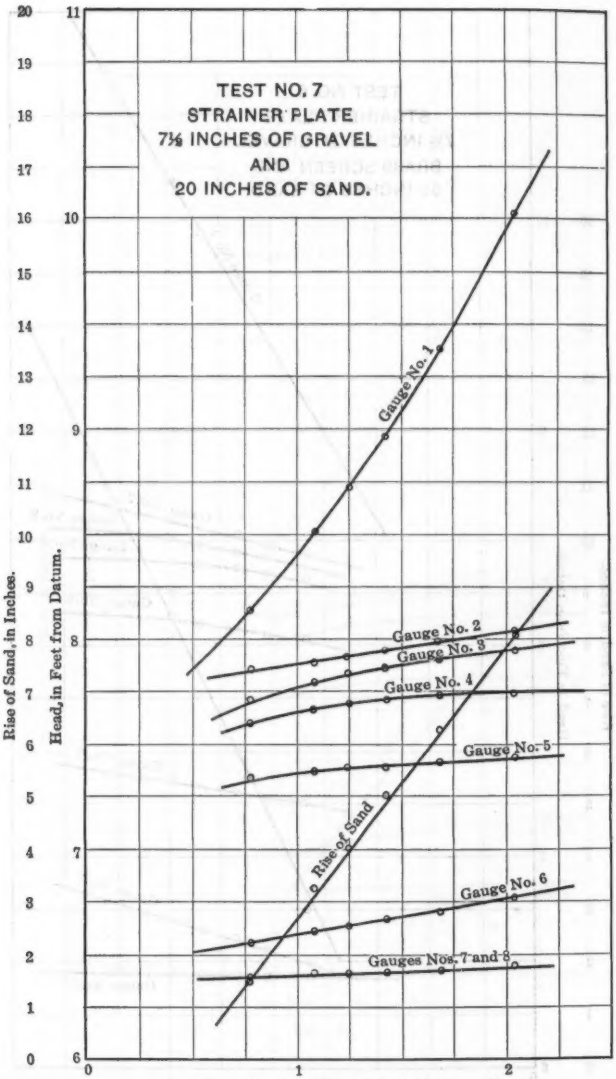

TEST NO. 7
STRAINER PLATE
7½ INCHES OF GRAVEL
AND
20 INCHES OF SAND.
Gauge No. 1
Gauge No. 2
Gauge No. 3
Gauge No. 4
Gauge No. 5
Rise of Sand
Gauge No. 6
Gauges Nos. 7 and 8
Rise of Sand, in Inches.
Head, in Feet from Datum.
Rate of wash-Vertical Rise, in Feet per Minute.

FIG. 10.

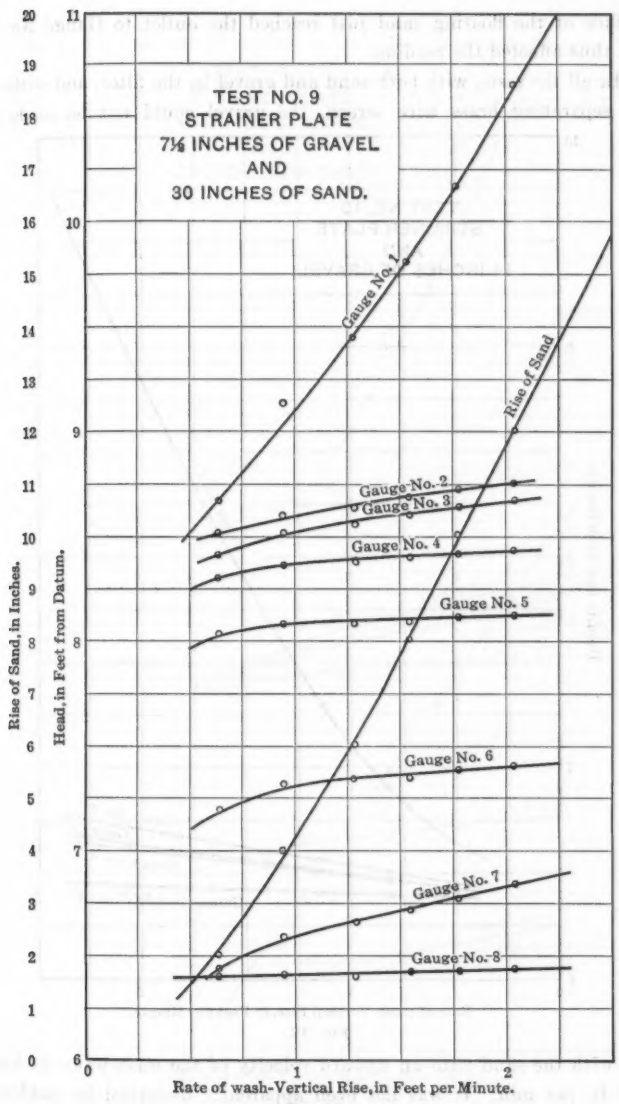

TEST NO. 9
STRAINER PLATE
7½ INCHES OF GRAVEL
AND
30 INCHES OF SAND.
Gauge No. 1
Rise of Sand
Gauge No. 2
Gauge No. 3
Gauge No. 4
Gauge No. 5
Gauge No. 6
Gauge No. 7
Gauge No. 8
Rise of Sand, in Inches.
Head, in Feet from Datum.
Rate of wash-Vertical Rise, in Feet per Minute.

FIG. 11.

surface of the floating sand just reached the outlet to Gauge No. 7, and thus affected the reading.

In all the tests, with both sand and gravel in the filter, and without the separating brass wire screen, the gravel could not be made to

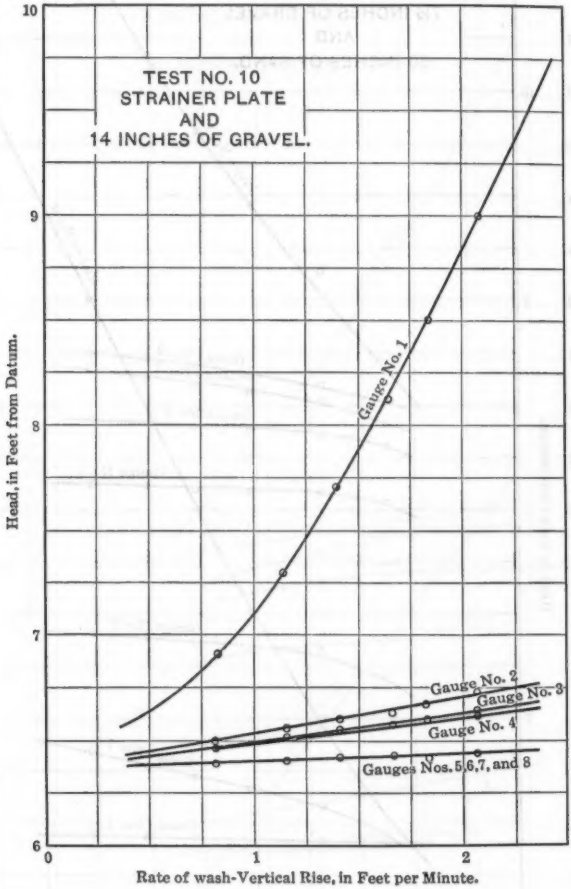


FIG. 12.

mix with the sand with an upward velocity of the wash-water as high as 2 ft. per min. It was not even apparently disturbed by suddenly applying the wash-water to the drained filter bed at a rate of 2 ft.

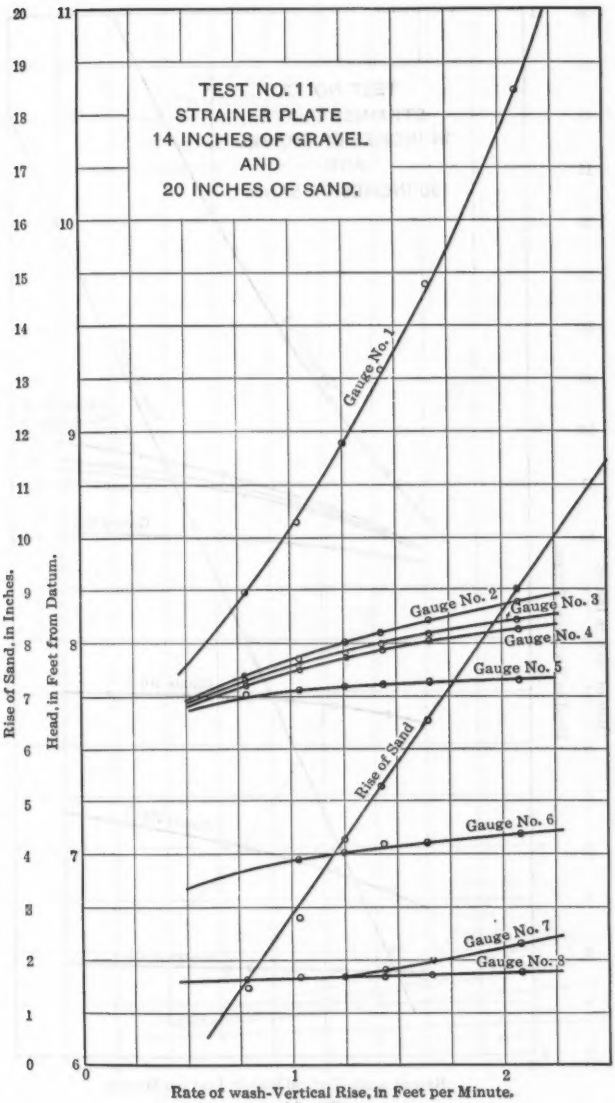

TEST NO. 11
STRAINER PLATE
14 INCHES OF GRAVEL
AND
20 INCHES OF SAND.
Gauge No. 1
Gauge No. 2
Gauge No. 3
Gauge No. 4
Gauge No. 5
Rise of Sand
Gauge No. 6
Gauge No. 7
Gauge No. 8
Rise of Sand, in Inches.
Head, in Feet from Datum.
Rate of wash-Vertical Rise, in Feet per Minute.

FIG. 13.

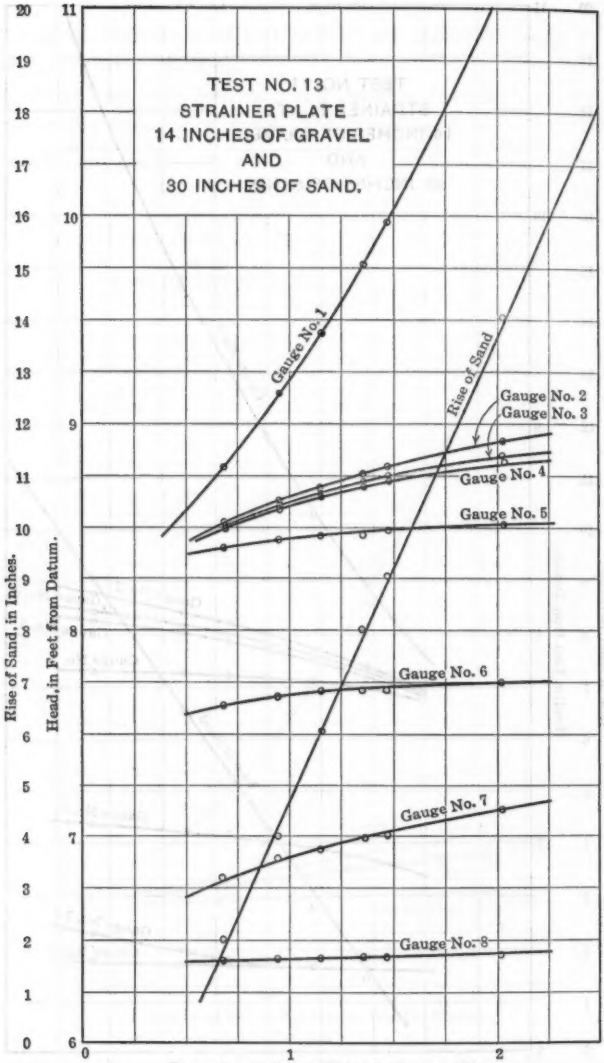

TEST NO. 13
STRAINER PLATE
14 INCHES OF GRAVEL
AND
30 INCHES OF SAND.
Gauge No. 1
Rise of Sand
Gauge No. 2
Gauge No. 3
Gauge No. 4
Gauge No. 5
Gauge No. 6
Gauge No. 7
Gauge No. 8
Rise of Sand, in Inches.
Head, in Feet from Datum.
Rate of wash-Vertical Rise, in Feet per Minute.

FIG. 14.

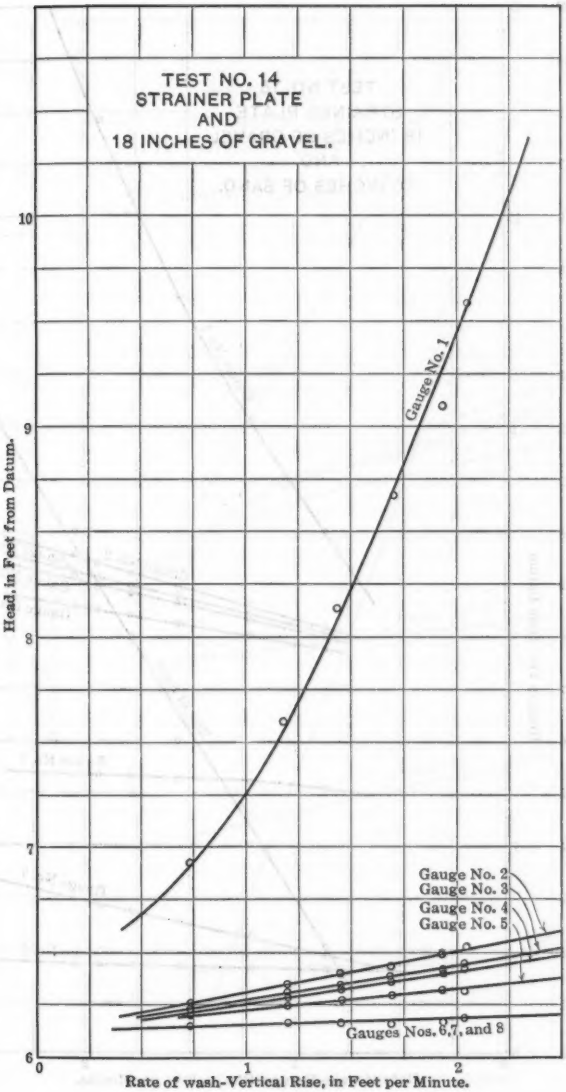

TEST NO. 14
STRAINER PLATE
AND
18 INCHES OF GRAVEL.
Head, in Feet from Datum.
10
9
8
7
6
0
1
2
Gauge No. 1
Gauge No. 2
Gauge No. 3
Gauge No. 4
Gauge No. 5
Gauges Nos. 6, 7, and 8
Rate of wash-Vertical Rise, in Feet per Minute.

FIG. 15.

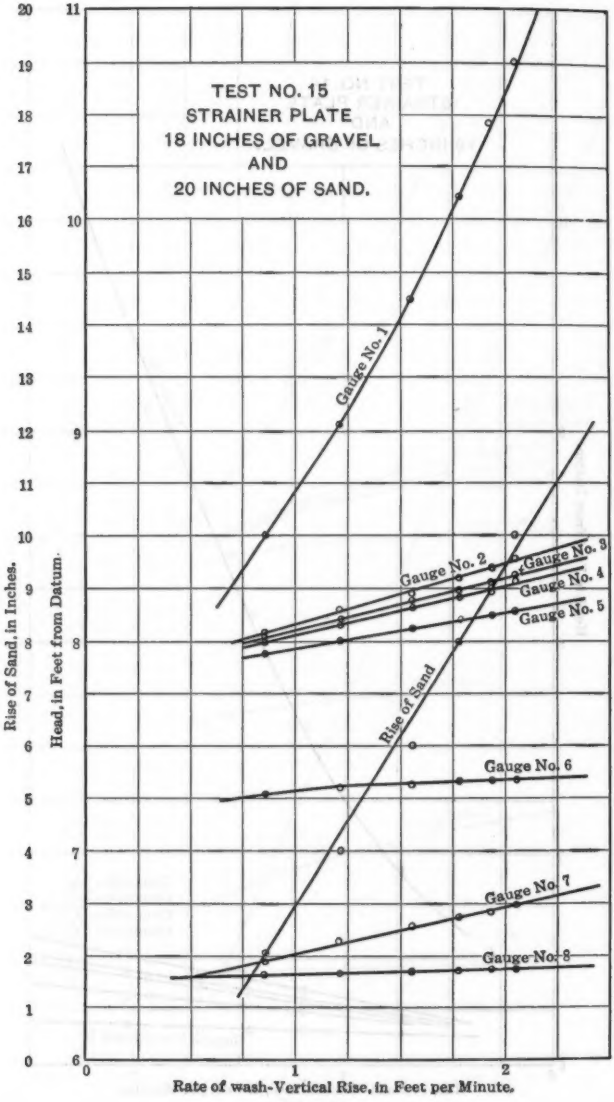

TEST NO. 15
STRAINER PLATE
18 INCHES OF GRAVEL
AND
20 INCHES OF SAND.
Gauge No. 1
Gauge No. 2
Gauge No. 3
Gauge No. 4
Gauge No. 5
Gauge No. 6
Gauge No. 7
Gauge No. 8
Rise of Sand
Rise of Sand, in Inches.
Head, in Feet from Datum.
Rate of wash-Vertical Rise, in Feet per Minute.

FIG. 16.

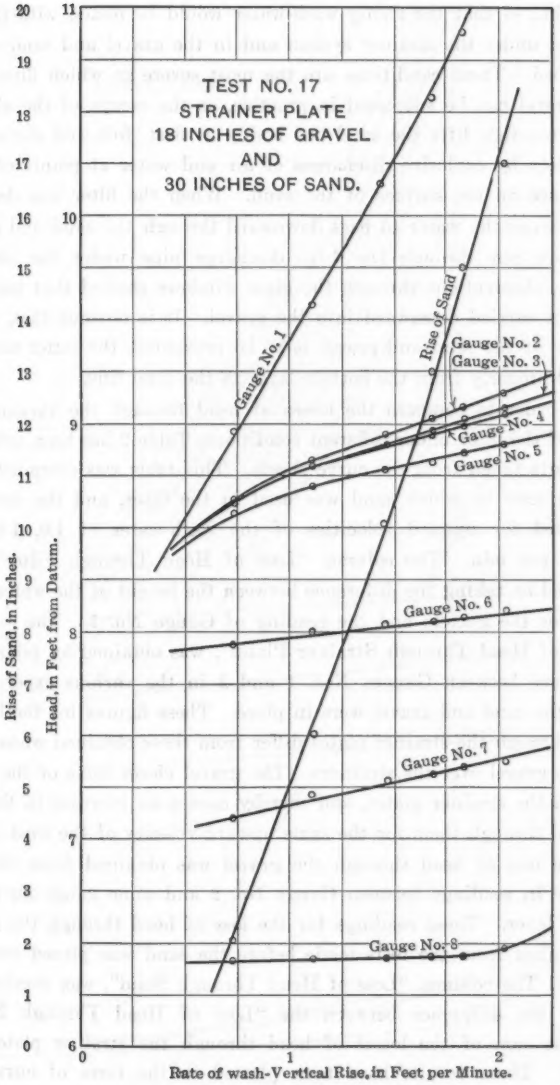

TEST NO. 17
STRAINER PLATE
18 INCHES OF GRAVEL
AND
30 INCHES OF SAND.
Gauge No. 1
Rise of Sand
Gauge No. 2
Gauge No. 3
Gauge No. 4
Gauge No. 5
Gauge No. 6
Gauge No. 7
Gauge No. 8
Rise of Sand, in Inches.
Head, in Feet from Datum.
Rate of wash-Vertical Rise, in Feet per Minute.

FIG. 17.

per min., so that the rising wash-water would be mixed with the air trapped under the strainer system and in the gravel and sand of the filter bed. These conditions are the most severe to which filter sand and gravel can be subjected in practice, as the escape of the air and water together lifts the sand bed bodily at first, followed almost immediately by explosive discharges of air and water at points of least resistance on the surface of the sand. When the filter was drained, by allowing the water to pass downward through the sand and gravel and flow out through the 4-in. discharge pipe under the strainer system, observations through the glass windows showed that the sand was not carried downward into the gravel. It is obvious that, if the mixing of the sand and gravel is to be prevented, the latter must be graded properly from the bottom layer to the sand line.

In order to compare the losses of head through the various portions of the filter under different conditions, Table 2 has been arranged with data taken from the curve sheets. This table was compiled from all the tests in which sand was used in the filter, and the data are tabulated for upward velocities of the wash-water of 1.0, 1.5, and 2.0 ft. per min. The column, "Loss of Head Through Filter", was obtained by taking the difference between the height of the water flowing over the gutters and the reading of Gauge No. 1. The column, "Loss of Head Through Strainer Plates", was obtained by taking the difference between Gauges Nos. 1 and 2 in the various experiments when the sand and gravel were in place. These figures for the loss of head through the strainer plates differ from those obtained when there was no gravel over the strainers. The gravel closes some of the openings in the strainer plates, and thereby causes an increase in the loss of head through them for the same upward velocity of the wash-water.

The loss of head through the gravel was obtained from the difference in readings between Gauge No. 2 and some gauge above the gravel layer. These readings for the loss of head through the gravel were taken from the tests made before the sand was placed over the gravel. The column, "Loss of Head Through Sand", was obtained by taking the difference between the "Loss of Head Through Filter" and the sum of the losses of head through the strainer plates and gravel. These results have been plotted in the form of curves on Fig. 18.

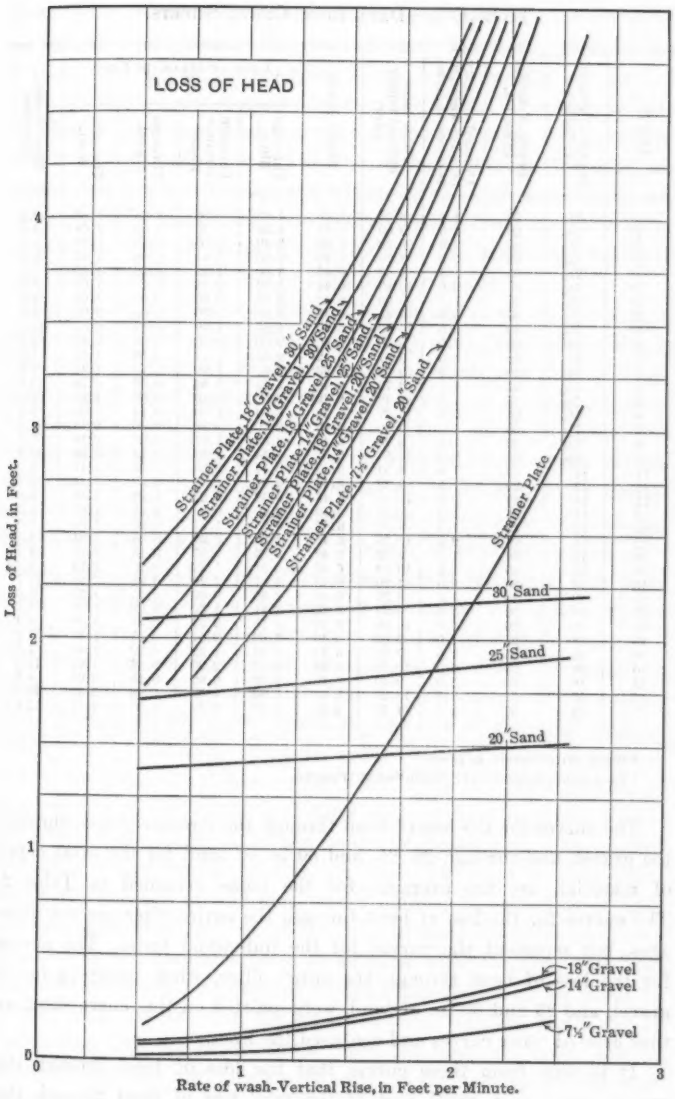

LOSS OF HEAD
Strainer Plate, 18" Gravel, 30" Sand
Strainer Plate, 14" Gravel, 30" Sand
Strainer Plate, 18" Gravel, 25" Sand
Strainer Plate, 14" Gravel, 25" Sand
Strainer Plate, 18" Gravel, 20" Sand
Strainer Plate, 14" Gravel, 20" Sand
Strainer Plate, 7½" Gravel, 20" Sand
Strainer Plate
30" Sand
25" Sand
20" Sand
18" Gravel
14" Gravel
7½" Gravel
Loss of Head, in Feet.
0
1
2
3
4
Rate of wash-Vertical Rise, in Feet per Minute.

FIG. 18.

TABLE 2.—DATA FROM CURVE SHEETS.

Test No.	Depth of gravel, in inches.	Depth of sand, in inches.	Upward velocity of wash-water, in feet per minute.	Total head applied under filter.	Static head.	Loss of Head, in Feet of Water.				Rise of sand, in inches.
						Through filter.	Through strainer plates.	Through gravel.	Through sand.	
4*	7½	20	1.0	8.33	6.40	1.93	0.58	0.06	1.29	2.5
4*	7½	20	1.5	9.00	6.42	2.58	1.20	0.09	1.29	4.9
4*	7½	20	2.0	9.92	6.43	3.49	2.05	0.14	1.30	7.3
5*	7½	25	1.0	8.70	6.40	2.30	0.60	0.06	1.64	2.8
5*	7½	25	1.5	9.42	6.42	3.00	1.22	0.09	1.69	6.3
5*	7½	25	2.0	10.29	6.43	3.86	2.03	0.14	1.69	9.8
6*	7½	30	1.0	9.00	6.40	2.60	0.60	0.06	1.94	2.6
6*	7½	30	1.5	9.81	6.42	3.39	1.30	0.09	2.00	6.6
6*	7½	30	2.0	10.63	6.43	4.20	1.99	0.14	2.07	10.7
7	7½	20	1.0	8.40	6.40	2.00	0.50	0.06	1.44	2.7
7	7½	20	1.5	9.07	6.42	2.65	1.12	0.09	1.44	5.3
7	7½	20	2.0	9.93	6.43	3.50	1.90	0.14	1.46	7.9
8	7½	25	1.0	8.75	6.40	2.35	0.50	0.06	1.79	3.7
8	7½	25	1.5	9.42	6.42	3.00	1.10	0.09	1.81	6.8
8	7½	25	2.0	10.27	6.43	3.84	1.90	0.14	1.80	9.8
9	7½	30	1.0	9.10	6.40	2.70	0.50	0.06	2.14	4.2
9	7½	30	1.5	9.76	6.42	3.34	1.07	0.09	2.18	7.8
9	7½	30	2.0	10.57	6.43	4.14	1.80	0.14	2.20	11.8
11	14	20	1.0	8.54	6.40	2.14	0.60	0.13	1.41	2.9
11	14	20	1.5	9.38	6.42	2.96	1.32	0.20	1.44	5.7
11	14	20	2.0	10.41	6.43	3.98	2.22	0.28	1.48	8.5
12	14	25	1.0	8.80	6.40	2.40	0.60	0.13	1.67	3.7
12	14	25	1.5	9.70	6.42	3.28	1.27	0.20	1.81	7.5
12	14	25	2.0	10.75	6.43	4.32	2.20	0.28	1.84	11.4
13	14	30	1.0	9.22	6.40	2.82	0.57	0.13	2.12	4.7
13	14	30	1.5	10.03	6.42	3.61	1.22	0.20	2.19	9.2
13	14	30	2.0	11.10	6.43	4.67	2.20	0.28	2.19	13.7
15	18	20	1.0	8.70	6.40	2.30	0.62	0.15	1.53	3.0
15	18	20	1.5	9.53	6.42	3.11	1.32	0.22	1.57	6.2
15	18	20	2.0	10.58	6.43	4.15	2.23	0.32	1.60	9.4
16	18	25	1.0	9.02	6.40	2.62	0.64	0.15	1.83	4.1
16	18	25	1.5	9.87	6.42	3.45	1.33	0.22	1.90	7.7
16	18	25	2.0	10.92	6.43	4.49	2.20	0.32	1.97	11.3
17	18	30	1.0	9.40	6.40	3.00	0.62	0.15	2.23	4.9
17	18	30	1.5	10.25	6.42	3.83	1.30	0.22	2.31	10.2
17	18	30	2.0	11.22	6.43	4.79	2.10	0.32	2.37	15.6†

* Brass wire screen in place

† Fine sand passed over wash-water troughs.

The curves for the loss of head through the strainer plates, through the gravel, and through 20, 25, and 30 in. of sand for the same depth of material, are the averages for the losses recorded in Table 2. The curves for the loss of head through the entire filter are not averages, but represent the curves for the individual tests. The curves for the loss of head through the entire filter, when using 7½ in. of gravel, and 25 and 30 in. of sand, were omitted on this curve sheet, as they crossed other curves and confused the lines.

It is seen from these curves that the loss of head through the gravel was a very small part of the total loss of head through the

filter. With an upward velocity of the wash-water of 2 ft. per min., the loss of head through the strainer plate was 2 ft.; through the 30 in. of sand, the loss of head was 2.2 ft., when washing at a rate of 2.0 ft. per min. From a study of these curves, it is interesting to note the effect on the loss of head through the sand of changes in the upward velocity of the wash-water. The loss of head through the sand increased very slowly as the upward velocity of the wash-water increased. The wash-water pushed the sand grains farther apart as the velocity increased, and more water flowed through the sand bed, but with very little increase in the loss of head through the sand.

Fig. 19 has four curves, three of which are from experiments in which the sand had an effective size of 0.41 mm. The other curve is taken from another set of experiments in which was used a 30-in. depth of sand which had an effective size of 0.31 mm. (Fig. 20 shows the mechanical analysis of this sand.) These curves represent the average of the distances obtained in the various tests between the top of the floating sand during the process of washing and the normal level of the sand when compacted during the process of draining. They also show that the height of flotation of sand having the same effective size, and subjected to the same upward velocity of the wash-water, was proportional to the depth of the sand. An analysis of the coarser sand used in these experiments is shown by Fig. 3.

When using the finer grade of sand in the filter, the height to which the sand floated was greater than that obtained with the coarser sand. This is shown by the curve for the rise of the sand with an effective size of 0.31 mm. With an upward velocity of the wash-water of 2 ft. per min., this difference amounted to 4½ in. On a number of tests with the large 4 000 000-gal. filter units at the Cincinnati Filtration Plant, a rise of 20 in. of the sand was obtained with an upward velocity of the wash-water of 2 ft. per min. The sand in these filters has an effective size of 0.39 mm., and a uniformity coefficient of 1.35. The measurement of the rise of the sand in the large filter units was made during the regular process of washing, after the filter had been removed from service on account of the accumulation of sediment in the sand bed. In this case the sand was dirty, whereas, in the set of experiments with the experimental filter, the sand was clean, due to the fact that it was continuously subjected to the washing process with filtered water, and was not used to filter water containing sediment.

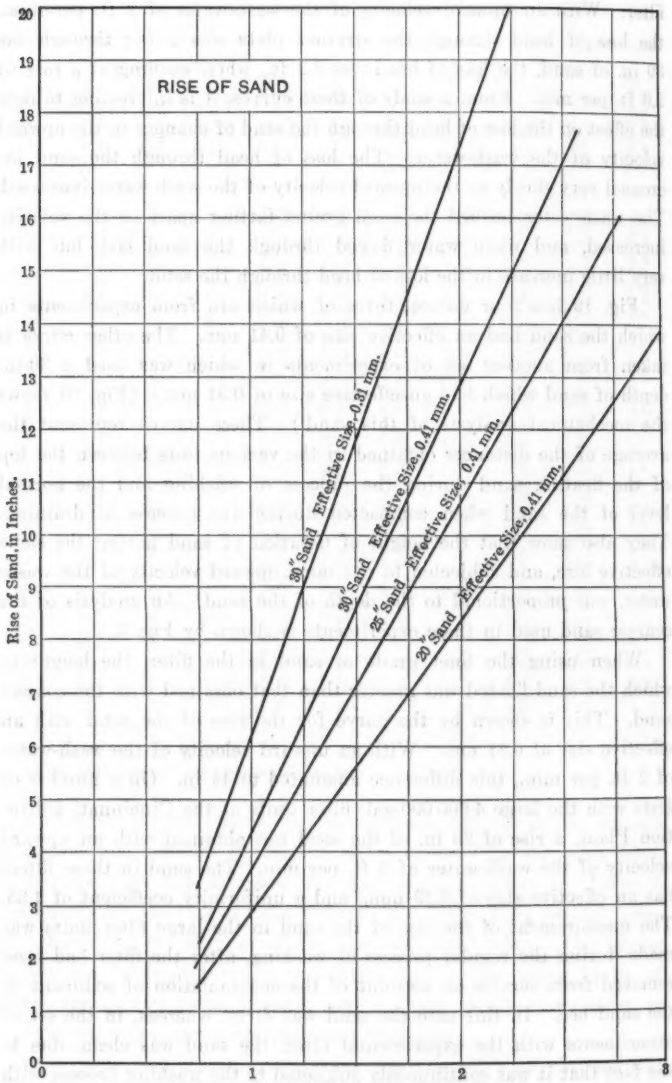

RISE OF SAND
Rise of Sand, in Inches.
Rate of wash-Vertical Rise, in Feet per Minute.
36" Sand Effective Size, 0.31 mm.
30" Sand Effective Size, 0.41 mm.
25" Sand Effective Size, 0.41 mm.
20" Sand Effective Size, 0.41 mm.
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20

FIG. 19.

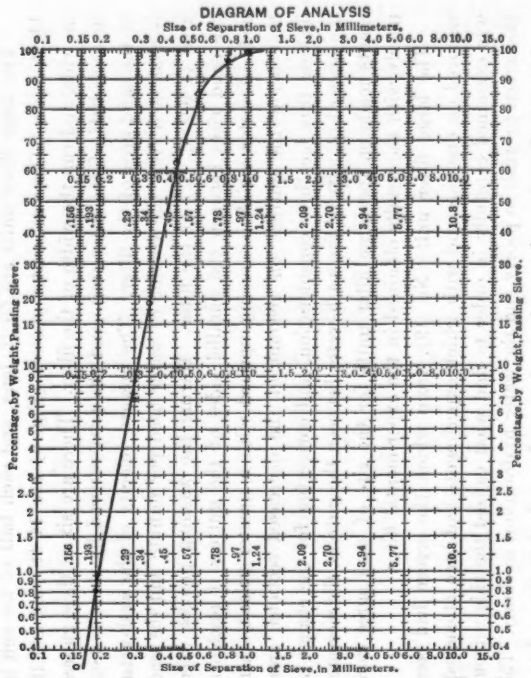


FIG. 20.

CINCINNATI, OHIO

MECHANICAL ANALYSIS OF FILTERING MATERIAL

No. of Sample EXPERIMENTAL FILTER

Location of Deposit AVERAGE OF 30" DEPTH (ALL)

Date of Collection MAY 16, 1913

Date of Analysis MAY 16, 1913

Remarks

Size of Separation of Sieve No.	Mm.	Weight Passing Sieve, in Grammes	Percentage, by Weight, Passing Sieve
2	10.8		
4	5.77		
6	3.94		
8	2.70		
10	2.09		
16	1.24		
20	0.97	98.0	98.0
24	0.78	95.3	95.3
30	0.57	85.6	85.6
40	0.45	62.3	62.3
50	0.34	19.5	19.5
60	0.29	7.0	7.0
80	0.193	.8	.8
100	0.156	.3	.3

Amount examined 100 Grammes

Effective Size 10 per cent finer than 0.31 mm.

60 per cent finer than 0.45 mm.

Uniformity Coefficient 1.45

Per cent finer than 0.20 mm =

" " " " 0.25 mm =

" " " " 0.85 mm =

OBSERVATIONS ON THE ORIGINAL AND ON TWO RECONSTRUCTED FILTER UNITS OF THE CINCINNATI FILTER PLANT.

The large filter units are 50 ft. long, and each half of the unit is 14 ft. wide. The wash-water gutters (Fig. 6) are 14 ft. long and 18 in. wide, and run the width of the filter. There are six of these gutters in each half of the filter unit, and the area covered by them is 18% of the sand area of the filter. The upward velocity of the wash-water is computed from the volume of water used per minute and the sand area (1 400 sq. ft.) of the filter. During the washing process, when the sand rises between the parallel sides of the gutters it reaches a body of water which is moving upward with a greater velocity than that below the gutters, on account of the decreased sectional area. Taking this fact into consideration, and correcting for the change in the upward velocity of the wash-water between the gutters, the measurements made on the large filter units for the rise of sand agree more nearly with the results obtained in the experimental filter.

The large filter unit (No. 19), from which the screen had been removed on one side and with which experiments had been attempted, was examined carefully in order to find what caused the side without the screen to wash so poorly and the water to drain out so slowly. The trouble was found in the strainer plates. In replacing these plates, workmen had used a cloth in some places to wipe off the cement from the top of the plates. This had forced some of the wet cement into the small openings of the strainer plates, where it had hardened and thus plugged them up. After cleaning out the holes in the plates, both sides of this filter were rebuilt without using the brass screen, 14 in. of gravel being used and placed as recorded in Test No. 10.

The original sand was placed on top of the gravel, which gave a depth of 30 in. to the sand layer. When the filter was washed with water having an upward velocity of 2 ft. per min., the sand flowed over the edges of the gutters. In order to prevent the loss of sand. the maximum upward velocity of the wash-water was reduced to 1.5 ft. per min., when estimated on the basis of the sand area. Fig. 6 shows the heights to which the main body of the sand floated in the 4 000 000-gal. filter units with upward velocities of the wash-water of 1.5 and 2 ft. per min.

Loss of sand over the edges of the gutters took place in the large filter units when the main body of the sand floated farther below the

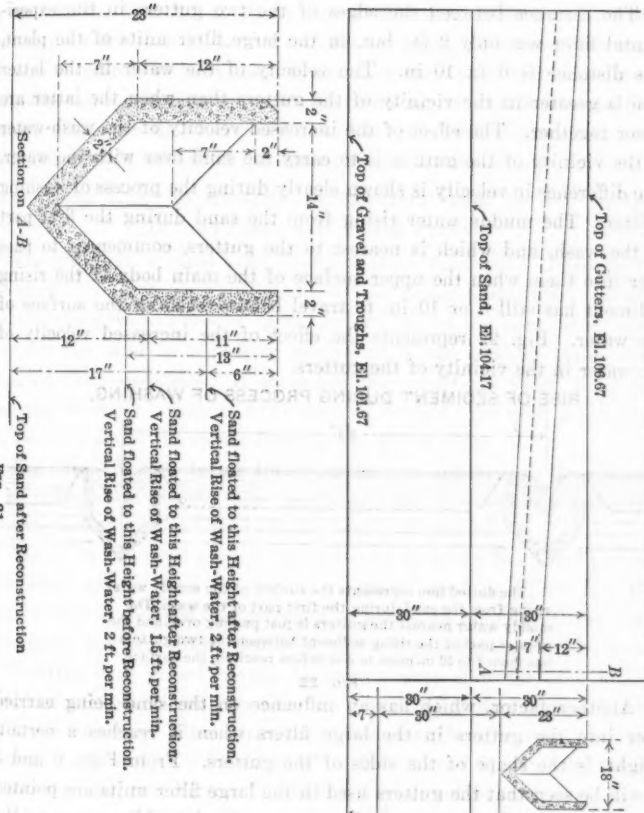
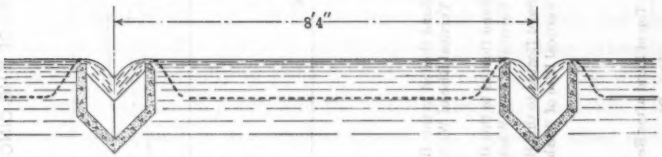

WASH-WATER GUTTERS OF THE CINCINNATI FILTER PLANT.
Top of Gutters, El. 106.67
Top of Sand, El. 104.17
Top of Gravel and Troughs, El. 101.67
30″
30″
7″
12″
A
B
30″
30″
7″
30″
30″
23″
18″
Top of Sand after Reconstruction.
Top of Sand before Reconstruction.
Top of Gravel after Reconstruction.
Top of Gravel before Reconstruction.
23″
7″
12″
2″
2″
7″
2″
14″
2″
12″
11″
13″
17″
6″
Section on A-B
Sand floated to this Height after Reconstruction.
Vertical Rise of Wash-Water, 2 ft. per min.
Sand floated to this Height after Reconstruction.
Vertical Rise of Wash-Water, 1.5 ft. per min.
Sand floated to this Height before Reconstruction.
Vertical Rise of Wash-Water, 2 ft. per min.
Top of Sand after Reconstruction

FIG. 21.

gutter edges than it did in the experimental filter. The reason for this is discussed in a subsequent paragraph.

The distance between the edges of the two gutters in the experimental filter was only 2 ft., but, in the large filter units of the plant, this distance is 6 ft. 10 in. The velocity of the water in the latter case is greater in the vicinity of the gutters than when the latter are closer together. The effect of the increased velocity of the wash-water in the vicinity of the gutters is to carry the sand over with the water. The difference in velocity is shown clearly during the process of washing a filter. The muddy water rising from the sand during the first part of the wash, and which is nearest to the gutters, commences to pass over into them when the upper surface of the main body of the rising sediment has still 8 or 10 in. to travel before reaching the surface of the water. Fig. 22 represents the effect of the increased velocity of the water in the vicinity of the gutters.

RISE OF SEDIMENT DURING PROCESS OF WASHING.



The dotted line represents the surface of the muddy water rising from the sand during the first part of the wash. The muddy water nearest the gutters is just passing over, and the greater part of the rising sediment between the two gutters has from 8 to 10 in. more to rise before reaching the surface.

FIG. 22

Another factor, which has an influence on the sand being carried over into the gutters in the large filters when it reaches a certain height, is the shape of the sides of the gutters. From Figs. 6 and 7 it will be seen that the gutters used in the large filter units are pointed at the bottom, and that they have perpendicular sides, whereas the sides of the gutters in the experimental filter sloped at an angle of 45 degrees. During Test No. 17 with the experimental filter, some of the sand passed over the gutters and the main body of the sand remained about 0.5 ft. below their tops. It could be seen that the sand which floated into the gutters came from the main body of the sand immediately below their outer edges, and through the zone of highest velocity of the wash-water. The material which escaped was only the finest sand particles, or silt.

Filter No. 5 was rebuilt, using 7½ in. of gravel and 30 in. of sand, but without the brass wire screen. The 7½ in. of gravel were placed as recorded in Test No. 2. Both filters—Nos. 5 and 19—have been in service for more than a year. From time to time, holes have been dug at various places in both these filters, and they have been examined carefully to see whether the sand was working its way down into the gravel. Thus far, they have been found to be in perfect condition, and their behavior in operation has been entirely satisfactory.

Conclusions Reached Relative to Reconstruction of All Filter Beds in Cincinnati Filter Plant.

The gravel in a rapid sand filter has three functions: It supports the sand layer; acts as a distributor of the rising wash-water, thereby making the upward pressure more uniform under the sand layer; and prevents the sand from passing downward to the strainer plates and clogging their openings. If these are the conditions which must be met, the practical questions to be decided relate to the total depth and the proper sizes of the stones forming the gravel layer.

The data collected during these experiments appeared to the writers to be sufficient to answer these questions, and, in consequence, the reconstruction of all the filter beds has been undertaken.

The 7½ in. of gravel used in the tests and in Filter No. 5 had proved successful, as also had the 14 in. of gravel used in Filter No. 19. Gravel, as it is pumped and screened on barges in the Ohio River near Cincinnati, contains all the material that will pass through a 2-in. hole.

Starting with gravel having an approximate diameter of 2 in., it would be possible to build an effective gravel bed with a depth of only 5 in. In the large filter units, it is necessary for the men who are placing the gravel to walk on it while they spread it out with rakes. It was decided, therefore, to build the gravel bed so deep that there would be no trouble due to the mixing of the different layers of gravel by the men walking on it, and the 14 in. of gravel used in Test No. 10 was decided on as giving a sufficient factor of safety in this respect.

In any gravel layer, acting as a support for the sand of a filter, there must be, of course, a rather uniform gradation from the coarsest to the finest particles, in order that the penetration of the smaller stones of the layer of gravel above into the voids at and near the surface of the layer below shall not be too great. This fact was taken into consideration in selecting the screens used for separating the

various sizes of gravel desired. Consideration was also given to the various sizes of gravel which could be obtained by screening washed Ohio River gravel as it was commonly sold, so that as little waste of material as possible should result.

The sand layer placed on the 4-in. gravel layer which had passed a ¼-in. mesh screen is washed in the filter in the usual way. This produces a gradation of its particles which merges them into the gravel layer beneath and thus forms a uniformly graded whole, commencing at the strainer plate with the 2-in. stones and ending with the finest sand at the surface of the bed.

The estimated cost of renewing the brass wire screen and of placing the filters of the Cincinnati Filter Plant in the same condition as that in which they were originally was more than $15 000. To have improved on the imperfect method of fastening the screen would have probably cost from $5 000 to $10 000 additional, or a total of from $20 000 to $25 000, depending on the improved method adopted.

From carefully kept cost records of the various operations required in rebuilding the filter beds, and including the cost of extra gravel, which is obtained for $1.39 per ton, the average cost for reconstructing each filter bed is $280. This makes the total cost of rebuilding the beds $7 854, to which should be added $650 for half the time of a foreman in supervising the work. The total cost, therefore, will be approximately $8 500, as compared with $20 000 or $25 000 for a renewal of the brass wire screen and improved methods of fastening it. As corrosion of the screen was inevitable, and as, by the substitution of a deeper gravel bed for the screen, the elimination of the latter was found to be possible, it seems that the problem has been satisfactorily solved.

Conclusions.

The conclusions reached as a result of the experimental work and of the writers' observations of the large filters of the Cincinnati plant may be briefly summarized as follows:

1.—In washing a rapid sand or mechanical filter, at rates of from 1.0 to 2.0 ft. vertical rise of the wash-water per minute, the greatest losses of head occur in the strainer system and in the sand.

2.—The loss of head through the strainer system used (which had openings equal to 0.3% of the sand area of the filter) was never greater than 2.22 ft., even with a velocity of the wash-water of 2.0 ft. per min.

With other forms of strainer systems and with different areas of openings the losses of head would differ, of course, from those found, even with the same velocities of wash-water.

3.—The loss of head in the gravel layer was found to be less than ½ ft. even with a velocity of the wash-water of 2.0 ft. per min. and with a gravel layer 18 in. deep.

4.—The loss of head through the sand did not exceed 2.37 ft. with a depth of 30 in. of sand and with a rate of flow of the wash-water of 2.0 ft. vertical rise per minute.

5.—The height of flotation of the sand during the washing process is affected by the depth of the sand, by its fineness, and by the velocity and uniformity of distribution of the wash-water. It is proportional apparently to the depth of the sand bed, other conditions being equal. Loss of sand over the edges of the waste troughs, during the washing process in the Cincinnati filters, was found to be influenced by the increased velocity of the wash-water, produced by the submergence of the gutters in the rising wash-water and by the distance of the floating sand from the bottom and sides of the gutters. Submergence of the gutters in the floating sand is undesirable.

6.—It was found that a properly graded layer of gravel would act as a distributor of the rising wash-water; and that layers as deep as 18 in. produced very low losses of head. Gravel layers 14 in. deep, and graded carefully in size from stones 2 in. in diameter at the bottom to particles that had passed through a ¼-in. mesh sieve at the top, were adopted as affording a sufficient margin of safety against improper grading in placing, or disturbance by wash-water moving at high velocity through the gravel.

7.—It was found possible, by introducing the deeper gravel layers in the Cincinnati filters, to do away with the brass wire screen originally placed between the gravel and sand, and which, although it provided at first a positive method of preventing the gravel and sand layers from becoming mixed, developed in the course of two or three years a mechanical weakness, due to imperfect methods of fastening and to disintegration of the brass itself by corrosion, thereby making it useless.

8.—The cost of renewal and of improving the method of fastening the brass wire screen in the Cincinnati filters was estimated to be nearly two and one-half to three times the cost of rebuilding the filters with deeper gravel beds, and the latter method, therefore, was adopted.

DISCUSSION

Mr. Fuller.

GEORGE W. FULLER,* M. AM. SOC. C. E. (by letter).—The authors of this paper have presented interesting and instructive data relative to important aspects of cleaning mechanical filters, especially with the aid of construction details, adopted not only at the Cincinnati plant, but also for those at New Orleans, Louisville, Minneapolis, Grand Rapids, and Evanston.

One of the difficulties which the screen at the top of the gravel in the original design was supposed to correct was doubtless caused by the jet action of the entering wash-water. In consequence of this action, the upper layers of the smaller gravel were seriously disturbed when the total thickness of the layer of gravel was no greater than in the original construction at Cincinnati. The effect of the deeper gravel layers, as interpreted by the writer, is essentially to prevent jet action and thus effect a substantial elimination of the disturbance more or less irregularly of the finer upper gravel layers.

In the earlier designs at Cincinnati, New Orleans, and Louisville, very little trouble with this type of strainer system design seems to have been caused by the quality of the metal, which was what might be briefly designated as "common brass". In the later designs, particularly at Minneapolis and Grand Rapids, effort was made in the direction of improvement, and bronze, instead of brass, was specified for the perforated plates at the bottom of the gravel and the bolts for holding these plates in place. Under the same specifications, practically speaking, serious trouble from breakage of plates and bolts resulted at Minneapolis; but at Grand Rapids there was practically no trouble, as was the case at Cincinnati, New Orleans, and Louisville. At Evanston, difficulties arose, not with the plates, but with the bronze bolts, and Monel metal was finally substituted for these.

In more recent designs it has been the writer's custom to use, not this type of strainer system, but perforated pipe in which the holes are staggered on the lower quadrant. This allows the wash-water to enter the filter in a downward direction about 45° from the horizontal, with the result that the entering stream strikes the filter floor and is deflected upward as a broken stream substantially free from jet action. This type of strainer system was probably first used by James H. Fuertes, M. Am. Soc. C. E., at Harrisburg, Pa., in 1905. The writer used this perforated pipe strainer system with success in a plant built in Burlington, Vt., in 1907, and in another plant, built at about that time, for the Botany Worsted Mills, Passaic, N. J. So far as known, no trouble has been experienced in any of these plants, but it is probably wise to provide somewhat deeper gravel layers than was

* New York City.

the custom in 1905 when the original design for the Cincinnati filters was made. Mr. Fuller.

It is worthy of note that, in the Ira H. Jewell *vs.* Minneapolis filter patent suit, the United States District Court at Minneapolis, in August, 1915, decided that the brass wire screen, such as used in the Cincinnati filters, infringes Claim No. 14 of Letters Patent No. 649 411 which is adjudged valid.

F. H. STEPHENSON,* M. AM. SOC. C. E. (by letter).—It was the writer's privilege to be closely connected with the construction of the Cincinnati Filter Plant, both before and after the filters were put into service. While the wire screens were being placed, certain predictions were made relative to their life, which, according to the prophets, ranged from 3 months to 5 years. Some difficulties were encountered in trying to fasten the screens securely to the ridges and, at the same time, keep the screens stretched tightly over the gravel. It took 40 man-hours to place the screens in one filter unit. Mr. Stephenson.

The writer has been converted to the idea of not being too insistent about keeping down the uniformity coefficient of the sand, and thinks the authors have shown that the larger particles settle out at the first wash and mingle with particles of similar size in the gravel layer. Some of the analyses made when the sand was being placed in the filters show an effective size of 0.33 mm. and a uniformity coefficient of 1.60. The writer recalls that the filters were washed and the top ½ in. of sand scraped off and wasted. That the sand in the filters now has an effective size of 0.39 mm. would indicate that some of the finer particles had been carried off during the many washings, even though the tops of the wash-water gutters are 30 in. above the original sand line.

E. G. MANAHAN,† M. AM. SOC. C. E. (by letter).—The details given in this paper regarding the behavior of filter sand and gravel and the losses of head through them when wash-water is applied at various rates of flow are interesting and helpful, and the authors' conclusion to substitute for the wire screen originally put in an additional thickness of gravel is sound. Mr. Manahan.

Although such a screen will probably not be used in the future, further information from the authors as to one of the causes assigned by them for the failure of the screen, namely, corrosion, would be of value. If marked corrosion has occurred in the brass screen during its eight years of service, this fact is important, especially if a like amount of corrosion has taken place in the brass strainer plates. Deterioration of the screen would naturally be expected to result from wear due to the movement of adjacent particles of sand and gravel, rather than from corrosion.

* Cleveland, Ohio.
† Wilmington, Del.

Mr. Manahan.

With the wire screen omitted, the Cincinnati strainer system fulfills satisfactorily all requirements for such a system, except that, with an 8-in. layer of graded gravel, as originally constructed, the velocity of the wash-water as it leaves the orifices in the strainer plates is so high that it may slightly disturb the gravel layer and in time cause mixing of the sand and gravel. This velocity at the ordinary rate used in washing is about 11 ft. per sec., and if this should be reduced to 3 or 4 ft. per sec., it is the writer's opinion that no disturbance whatever of the gravel would take place.

Such a reduction in the velocity through the orifices could have been provided for at an expense no greater than the cost of the screen The cost of the screen and its fastenings was about $0.30 per sq. ft. of net filter area; that of the strainer plates and their fastenings, $0.40; that of placing the screen and plates, $0.33; and that of the concrete ridges in place, $0.20; total, about $1.23. These represent prices paid by the City, and are reasonable, except that the price for placing the screen and plates was too high and that for the concrete was too low.

For the cost of the screen it would have been possible to place a second strainer plate beneath the one used, making both plates, perhaps, a little thinner. The second plate would be of practically the same shape as the first, but placed in an inverted position, with the edges of the two plates in contact and so as to leave a diamond-shaped area between them. Then, by a proper choice of orifices in the plates, the greater part of the loss of head could be made to take place in passing through the lower plate, allowing the water to escape at a comparatively low velocity through numerous orifices in the upper plate. The holes would be arranged so that they would not come opposite one another.

This type of strainer system would give a control of the wash-water more nearly positive than that accomplished by using deep layers of gravel, which latter control is somewhat haphazard. However, the cost of the additional plate, although no greater than that of the screen, would considerably exceed, even at times of normal prices for brass, the cost of the additional gravel; although in a new plant this excess would be reduced to some extent by the cost of the extra depth of filter tank required to accommodate the gravel as well as by the cost of added strength in the filter floor and foundations.

Mr. Willcomb.

GEORGE E. WILLCOMB,* ASSOC. M. AM. SOC. C. E. (by letter).—This paper, presenting a scientific discussion of the many factors relating to the high-velocity wash of a rapid sand filter, is timely and instructive. The literature on the washing of rapid filters is meager. Most of the published reports are based on the individual conclusions of different operators arrived at without any extended research, and,

* Albany, N. Y.

Mr. Willcomb.

from necessity, they contain frequent instances of inaccuracy of interpretation. This paper is especially timely in that it tends to focus attention on the general subject of filter washing at a time when radical action on the question seems imminent. At a recent meeting of a western engineering society, the air-wash as applied to rapid filters was adversely criticized, and a member of the engineering staff of a State Board of Health took the extreme position of recommending that it be abolished in future plants. Such views as these emphasize the importance of research work in connection with filter washing, in order that such a useful adjunct to the washing process as air agitation may not be rejected without good reason. Eastern plants, operating with a water of low turbidity and high in color and sewage contamination, have found the air-wash to be most efficient, and plants constructed 12 years ago are having continued success with this means of agitation. It has been found that most of the troubles have been physical, that is, have been caused by faulty design of the air-wash system.

On the publication of the report of the Commissioners of Water-Works of Cincinnati, in 1909, in which the placing of screens at the junction of the sand and gravel was advocated, many filtration engineers of extended experience in the operation of rapid filters were skeptical as to the benefits to be obtained by the use of such screens at this point. To be sure, the condition brought about by the placing of such screens was almost ideal, for certainly no mixing of the sand with the gravel could result; yet, owing to the stresses that were likely to act on the screen at the beginning and end of a wash, and the likelihood of uneven distribution of the wash-water, all factors seemed to be against any permanency of alignment of the screen.

The ridge-block construction for filter bottoms, though old in principle, has been rapidly developed during the past 5 years, and varying modifications of this type have been incorporated in the design of several plants of recent construction. The advantages of this system are: a smaller volume of gravel is required; unwashed zones in the gravel medium are eliminated; and jet action of the incoming wash-water is prevented. The disadvantages are: increased cost and complication of the construction; the inherent weakness of the design tending toward shearing the anchor-bolts and thus letting sand into the collector channels; and the trouble involved in overhauling the beds during periods of rejuvenation.

As long as a screen for holding down the gravel was used, and only the trapezoidal channels were filled with gravel, there is no doubt that this system assured an even distribution of the wash-water and prevented unwashed zones; but, as soon as the retaining screen was done away with and the gravel layer increased above the tops of the chan-

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nels, dead spaces were introduced, and practically all the advantages of the system were lost.

From the extended experimentation, the grading of the gravel appears to be of the utmost importance. This paper proves conclusively that the depth and the arrangement of the sizes do not cause a loss of head of more than 0.3 ft. even with a 2-ft. vertical rise. The function of the gravel layer being to support the sand medium, distribute the wash-water, and prevent sand grains from entering the under-drain system, the writer has come to the conclusion that 14 in. of graded gravel, a simple type of sand-valve, and a flat floor, represent optimum conditions, both in regard to economy and good design. That portion of the gravel surrounding the sand-valves should be composed of large units, say from 2 to 3 in. in diameter, and should be at least 4 in. thick. The next layer ranges from 1 to 1½ in., and is 2 in. deep. Next should come a 3-in. layer, ranging from ¾ to ½ in. The next layer is 2½ in. thick, and runs from ½ to ¼ in., followed by a 2½-in. layer of ¼ to $\frac{1}{16}$-in. size. Gravel graded in this manner provides for adequate deflection of the wash-water jet without undue throttling, and allows of a much wider spacing of the sand-valves than is customary. The tendency of the wash-water is to follow up the surfaces of the large gravel units, and conditions are brought about that are quite akin to the mathematical placing of spherical bodies around the valves, a procedure that has been introduced with favorable results quite recently. The main factor to be considered in placing the gravel is the proper sizing of the various layers. Generally speaking, a layer of gravel will remain on top of a lower layer, provided it contains no particles less than one-third the size of the particles on which it rests. The gravel adjoining the sand should not be too coarse in relation to the sand, else mud-balls will tend to be deposited by a too sudden decrease of the velocity during the filtration period proper.

When the Albany experiments in double and rapid filtration were begun in 1905, data were available from the experimental plants at Philadelphia, Cincinnati, and Columbus. At this time the ridge-block under-drain system and high rates of washing were being exploited, and accordingly a small filter with an inverted cone depression at the under-drain was experimented with. A very simple cylindrical sand-valve was developed from seamless brass tubing of No. 20 stock, Stubb's steel wire gauge. The tubes were punched out of one piece with a die, and cost 9 cents apiece to manufacture. The tubes were 4 in. long and had a ⅝-in. bore. There were two rows of staggered holes, ¼ in. down from the top, each being 0.161 in. in diameter. Each hole had an area of 0.02036 sq. in., and the combined area of the eight openings amounted to 0.1629 sq. in., which was 0.2% of the entire filter surface. This was a smaller rate than usual, and gave rise to a friction loss of

head of 3.0 ft. when washing with a vertical rise of 1 ft. The bottom of the tube was made with a flare, and two sets of depressions were sunk into the vertical sides to form a bond when they were concreted into the filter bottom. When the valve was in place in the bed, the holes of the lower set were only ¼ in. above the filter floor, thus preventing the silting up of any portion of the gravel under-drains.

This small filter had a glass side through which the effects of the washing could be observed. The gravel layer consisted of 12 in. of selected pebbles, graduated from 3 in. to ⅛ in., and on this was placed 24 in. of a 0.32-mm. sand having a uniformity coefficient of 2.50. Different rates of washing were used up to 2 ft. rise, and no tendency of the sand to mix with the gravel was noted. On starting the wash, the entire sand body would rise for a distance of from 4 to 6 in., when the bottom particles would begin to detach themselves and take up the motion of the various currents. Great importance was attached to this phenomenon at the time, for it was thought that the violent breaking away of the sand particles would shear the strands of organic matter attached to the sand grains when the large filters were placed in operation, and thus aid materially in the washing process. It soon became apparent, however, that these conditions were not typical, owing to the restricted sand surface of the experimental filter, and that surface tension had a lot to do with the phenomenon, which would not occur in the large beds. The results of these experiments indicated that washes up to 2 ft. rise per min. could be used safely without admixture of the sand and gravel.

When the sixteen large units were built, flat floor bottoms were decided on, as the advantages of the bottom with pyramidal pockets did not justify their additional cost of construction. The results obtained with the experimental filters had demonstrated clearly that the rapid filters, operating as scrubbers without coagulation, would not wash the sand in place in the bed indefinitely, but would have to be given supplemental washes periodically by ejection or other means. As ejection meant removing all the sand down to the gravel, and as this process would be likely to disturb the gravel layer, it was deemed probable that the gravel might have to be regraded at intervals, and that the pryamidal depressions in the floor, in such a case, might interfere with the work.

The sand-valves developed in the experiments were used in the construction of the large beds. These valves were cast in small slabs outside the beds, and, when hardened, were set over trapezoidal ducts formed in the floors of the filters. These ducts ran across the beds and discharged into a main collector of 2 ft. diameter. The main collector and the under-drain ducts acted as a compensator for the wash-water, and insured equal distribution throughout the bed during the

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wash. The valves were set 9 in. on centers, and were covered with 1 ft. of gravel graded in four lots from 3 in. to $\frac{1}{16}$ in. A depth of 30 in. of silica sand, of 0.45 mm. effective size and a uniformity coefficient of 2.50, was eventually selected for the filter medium. The beds were divided into two parts by the central wash-water gullet which ran the long way of the beds. The wash-water troughs were of rectangular section, 6 in. deep and 10 in. wide (inside dimensions), and were 7.5 ft. apart. One-half of their top opening was closed on the discharge end, so that the maximum travel of any particle of suspended matter was about 4 ft. The crests of the wash-water troughs were placed 10 in. above the normal surface of the sand, and, during the wash, with a 1.5-ft. rise per min., the sand was elevated 6 in.

As long as the sand was new, no difficulty was experienced in washing the beds. At the rate of a 1.5-ft. rise, the entire sand medium was in suspension and agitation, and, after the expenditure of about 3% of the entire yield for wash-water, the beds were washed satisfactorily.

TABLE 3.

Section.	Before Washing. Effective size.	Uniformity coefficient.	Percentage of moisture.	Turbidity.	Organic nitrogen.	Oxygen consumed.
A—Top (0 to 10 in.)....	0.35	2.08	5.9	360	0.04	7.3
B— " " "	0.33	2.24	6.4	480	0.12	6.0
C— " " "	0.38	1.95	5.6	240	0.00	7.9
D— " " "	0.37	1.67	6.4	370	0.00	8.3
Average.....	0.36	1.98	6.1	360	0.04	7.4
A— (10 to 20 in.)....	0.36	2.37	5.2	105	0.00	5.5
B— " "	0.42	2.38	5.1	110	0.00	4.8
C— " "	0.38	2.23	5.3	75	0.00	6.2
D— " "	0.50	2.80	4.5	115	0.04	9.5
Average.....	0.40	2.44	5.0	100	0.01	6.5
A— (20 to 30 in.)....	0.54	2.77	5.5	130	0.00	4.4
B— " "	0.49	3.26	7.4	60	0.00	3.8
C— " "	0.40	3.00	5.7	120	0.00	5.6
D— " "	0.52	3.07	4.5	105	0.00	8.4
Average.....	0.49	3.02	5.8	104	0.00	5.6

After Washing. Section.	Turbidity.	Organic nitrogen.	Oxygen consumed.
Avge. *A* + *B* (0 to 10 in.)....	165	0.00	6.6
" *C* + *D* (0 to 10 in.)....	110	0.04	5.1
" *A* + *B* (10 to 20 in.)...	115	0.00	6.9
" *C* + *D* (10 to 20 in.)...	50	0.00	4.4
" *A* + *B* (20 to 30 in.)...	90	0.00	5.9
" *C* + *D* (20 to 30 in.)...	60	0.00	4.8

Efficiency of Washing. Percentage of Reduction.

Section.	Turbidity.	Organic nitrogen.	Oxygen consumed.
0 to 10 in.....	62.1	50.0	20.7
10 to 20 in.....	18.5	....	13.1
20 to 30 in.....	27.7	....	8.1

Table 3 shows the condition of the new sand in place in the bed after a month's run. Four stations were selected in the filter, and tests were made on 10-in. sections from top to bottom. For the

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chemical work, 100 grammes of the sand were shaken in the sand-engine with 100 c.c. of distilled water for 1 min.; then the whole was diluted to 1 liter, and the different determinations were made on the stock suspension. The newness of the sand is shown by the low percentage of moisture retained on the grains. No organic coats had been formed on the sand at this point, as is also shown clearly by the organic nitrogen and oxygen consumed factors. It is interesting to note the grading of the sand according to the hydraulic values of the grains. As the depth of sampling increases, the effective size increases in like proportion. This grading of the sand is much more marked in the case of a fresh sand than with an old one. There is less friction to the passage of the grains in the former case, and stratification is brought about at the conclusion of the wash. It will be noticed, however, that the uniformity coefficients of the bottom samples are high, denoting the presence of much fine material at the bottom of the bed. It has been found by experience that to accomplish complete stratification according to the hydraulic values of the grain is practically impossible. Convection currents and the liberation of entrained air during the washing process have more or less effect in defeating the attainment of this condition, especially in the case of old sand coated with organic growths.

TABLE 4.

Before Washing.						
Section.	Effective size.	Uniformity coefficient.	Percentage of moisture.	Turbidity.	Organic nitrogen.	Oxygen consumed.
A—Top (0 to 10 in.)	0.52	1.52	16.6	1 750	1.60	16.5
B— " " "	0.61	1.64	17.7	1 750	2.20	19.7
C— " " "	0.58	1.72	15.9	1 800	2.00	18.7
D— " " "	0.58	1.76	13.1	1 600	1.60	15.3
Average	0.57	1.66	15.7	1 725	1.90	17.6
A— (10 to 20 in.)	0.61	1.70	10.1	1 750	1.44	15.7
B— " "	0.60	1.80	9.7	1 400	0.96	15.5
C— " "	0.56	1.98	12.8	1 500	1.04	14.1
D— " "	0.55	1.83	9.3	1 200	0.88	9.0
Average	0.58	1.83	10.5	1 460	1.08	13.6
A— (20 to 30 in.)	0.60	2.00	8.3	1 200	0.64	12.3
B— " "	0.65	1.67	8.3	1 250	0.24	12.5
C— " "	5.55	1.88	10.8	1 400	0.72	11.7
D— " "	0.57	1.77	13.5	1 400	0.80	9.5
Average	0.59	1.82	10.2	1 320	0.60	11.5

After Washing.			
Section.	Turbidity.	Organic nitrogen.	Oxygen consumed.
Avge. *A* + *B* (0 to 10 in.)	1 600	1.80	17.3
" *C* + *D* (0 to 10 in.)	1 500	1.60	15.1
" *A* + *B* (10 to 20 in.)	1 400	0.96	14.5
" *C* + *D* (10 to 20 in.)	1 400	0.96	8.4
" *A* + *B* (20 to 30 in.)	1 350	0.24	13.7
" *C* + *D* (20 to 30 in.)	1 250	0.64	9.9

Efficiency of Washing.
Percentage of Reduction.

Section.	Turbidity.	Organic nitrogen.	Oxygen consumed.
0 to 10 in.	10.2	10.5	7.4
10 to 20 in.	3.6	11.1	16.2
20 to 30 in.	.....	25.1	15.7

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Table 4 shows the condition of the same filter after it had undergone a summer's ripening. The effective size of the sand had been increased by successive scrapings of fine sand from the surface. In considering these data it must be borne in mind that the filter was operating as a scrubber, filtering water which had received 12 hours of plain subsidence. It will be noted that stratification is no longer as pronounced. The organic factors have materially increased, as has the turbidity. A striking feature is the increase in the percentage of moisture, denoting sponge-like action on the part of the sand grains. The efficiency of the wash has materially lessened. In one case, at the bottom section of the bed, there is no efficiency. Everything points to a complete paralysis of the function of the wash. Examined through the microscope, it was observed that the grains were completely coated with brown mossy growths of sewage mould and fungi. Fruiting bodies were in evidence, and the strands of hyphæ were seen bound about several adjacent grains in such a way that the sand was consolidated into clots and mounds. Under such conditions as these, the wash-water would be ineffective at a 2-ft. or a 3-ft. vertical rise. The water, taking the course of least resistance, would pass around these masses of sand without disintegrating them, and, as a consequence, they would sink to the bottom of the bed and form the nucleus of a mud-ball or impervious zone.

Fig. 23 is a diagram of one of the rapid filters, with symbols to represent the condition of the sand. It will be noted that close to the wall the bed is badly silted up from top to bottom; the turbidity of the sand ranges from 2 000 to 3 000. Needless to say, these portions of the bed are inert, and neither filter nor wash. Adjacent to these mud-balls are mounds composed of sand of relatively high effective size and uniformity coefficient. It is at these points that the wash-water, diverted by the mud-balls, finally breaks through, and in so doing, carries much of the coarse gravel from the bottom of the bed to the surface, due to the greater velocity of the wash-water. If this happens in the proximity of a wash-water trough, sand is very likely to be carried into the sewer and lost. It is for this reason that the form or special devices attached to a trough are apt to be ineffectual, in the long run, in preventing the loss of sand. Theoretically, when the wash-water arrives at the point where the troughs are placed, the velocity is increased in direct proportion to the lessening of the superficial area by the surface of the troughs, and some sand is apt to be carried into them and lost. Some designers have attempted to correct this by modifying the shapes of the troughs, and one finds all shapes in use, from semicircular sections to rectangular ones. However, owing to abnormal conditions of the sand and the sudden release of entrained air, the velocity of the wash-water is bound to increase at times, and carry the sand into the trough, regardless of its shape.

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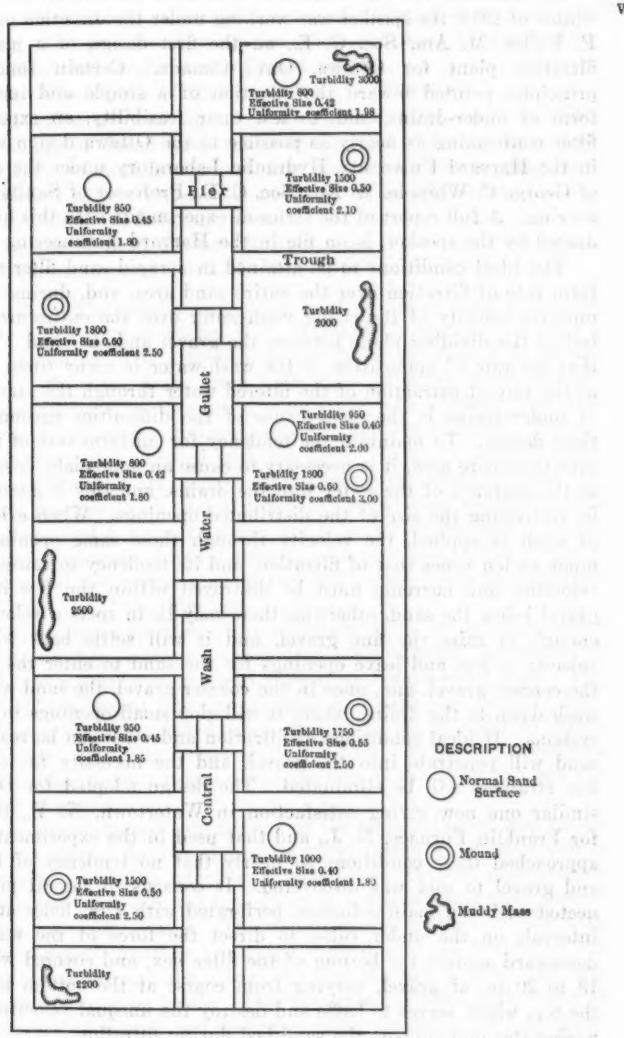

FIG. 23.

Mr. Pirnie.

H. MALCOLM PIRNIE,* JUN. AM. SOC. C. E.†—During the fall and winter of 1913, the speaker was working under the direction of Weston E. Fuller, M. Am. Soc. C. E., on the first design of a mechanical filtration plant for Ottawa, Ont., Canada. Certain fundamental principles pointed toward the adoption of a simple and inexpensive form of under-drains, and, to test their feasibility, an experimental filter conforming as nearly as possible to the Ottawa design was built in the Harvard University Hydraulic Laboratory under the direction of George C. Whipple, M. Am. Soc. C. E., Professor of Sanitary Engineering. A full report of the series of experiments with this filter, conducted by the speaker, is on file in the Harvard Engineering Library.

The ideal conditions to be attained in a rapid sand filter are: uniform rate of filtration over the entire sand area, and, during washing, uniform velocity of the rising wash-water over the entire area of the bed at the dividing plane between the gravel and the sand. The fact that the rate of application of the wash-water is many times as great as the rate of extraction of the filtered water through the same system of under-drains is the chief cause of the difficulties encountered in their design. To maintain the tendency for uniform rate of filtration over the entire area, it is necessary to cause an appreciable loss of head at the entrance of the water into the drains, and this is accomplished by restricting the size of the distributed openings. When a high rate of wash is applied, the velocity through these same openings is as much as ten times that of filtration, and its tendency to cause unequal velocities and currents must be destroyed within the few inches of gravel below the sand, otherwise there may be in spots a velocity high enough to raise the fine gravel, and it will settle back where the velocity is less and leave openings for the sand to enter the voids of the coarser gravel, and, once in the coarser gravel, the sand will easily wash down to the drains, where it will clog small openings in strainer systems. If ideal conditions of filtration and wash can be realized, no sand will penetrate into the gravel, and the necessity for expensive fine strainers will be eliminated. The design adopted for Ottawa, a similar one now giving satisfaction in Watertown, N. Y., that used for Franklin Furnace, N. J., and that used in the experimental filter, approached these conditions so nearly that no tendency of the sand and gravel to mix was discovered. It consists of lateral pipes connected with the main collector, perforated with ¼-in. holes at regular intervals on the under sides, to direct the force of the wash-water downward against the bottom of the filter box, and covered with from 18 to 20 in. of gravel, varying from coarse at the bottom to fine at the top, which serves to baffle and destroy the unequal velocities of the wash-water and support the sand bed during filtration.

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† Now Assoc. M. Am. Soc. C. E.

Mr. Pirnie.

The following is an extract from the report of the tests on the experimental filter. It describes the process of filtration as well as that of washing. One is dependent on the other, and must be observed in order to afford full explanation of what occurs.

Objects.—The objects of these tests were to determine:

1. The effect of washing on a mechanical filter built up with simplified forms and under-drains and different sizes and depths of sand and gravel, first with a water-wash alone, and second with a water-wash preceded by an air-wash;
2. The loss of head during washing;
3. The loss of head during filtration;
4. The intensity and depth of clogging in the sand bed.

Apparatus.—The mechanical filter used in these tests is in the Hydraulic Laboratory at Pierce Hall, Harvard University. As shown in Figs. 24 and 25, it is a wooden box, 2 ft. 11 in. wide, 6 ft. 1½ in. long, and 7 ft. 5 in. deep, fitted with glass windows, a galvanized-iron gutter discharging through a sluice-gate into a waste pipe, and the necessary piping for influent, effluent, and wash-water. An adjustable overflow is connected with the filter. In one side of the box at suitable elevations are placed gauge-cocks with straight tubes for registering positive head, and **U**-tubes for registering negative head. A gauge tube and a pressure gauge are attached to the under-drains. The effluent discharges through a gate and a pipe which is closed by a water seal. All the gauges read in feet and inches below an arbitrary zero set at the top of the filter box.

The under-drains are formed by six lateral 2-in. wrought-iron pipes screwed into a 3-in. collector which discharges through the center of the floor into a 4-in. wrought-iron effluent and wash-water pipe. The free ends of the lateral pipes are capped, and each pipe has five ¼-in. holes in its under side, placed as shown in Fig. 24, in order to direct the wash-water vertically downward to the floor. Between the laterals, and at the sides, wooden ridges direct the flow of the water through the first grade of gravel.

A ¼-in. pipe is tapped into the top of the main collector, and passes through the side of the box through a valve into a **T**, above which is another valve and below which is a waste pipe. A ¼-in. pipe extends from the upper valve into the top of a glass bottle set on a shelf several feet above the under-drains. The mouth of the bottle is closed with a rubber stopper. Another ¼-in. pipe, tapped into the bottom of the collector, passes through the side of the box, through a valve, and into the bottom of the glass bottle. This apparatus is filled with water at the beginning of a test, and indicates when air forms in the under-drains by allowing it to collect in the top of the bottle, water being forced out at the bottom.

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Both the waste-wash and effluent pipes discharge into a trough below the filter, at one end of which is a weir. This is shown in Fig. 26. The water passing from this weir falls into one or the other of two weighing tanks, according to the way the controlling lever is placed.

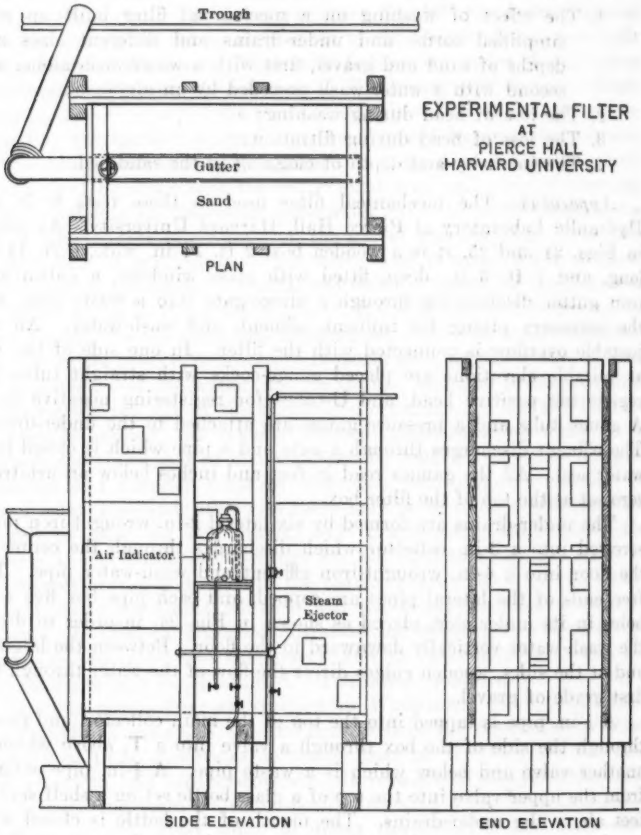

FIG. 24.

By these tanks the rate of wash and effluent discharge can be determined. The waste-valves at the bottoms of the tanks are controlled by levers above the weighing arms, and the discharge through them enters a concrete reservoir of considerable capacity in the sub-basement.

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A Blake duplex steam pump, Fig. 26, with suction pipes in this reservoir, lifts the water and discharges it into the bottom of an air-tight pressure tank. At a sump in the discharge pipe from the pump to the tank are connected a waste-gate, discharging into the reservoir, and a 4-in. supply pipe for the filter. A ½-in. air line, from the Westinghouse direct-acting air compressor, enters the bottom of the tank through a valve. This arrangement makes possible a pressure service of either air or water. On the supply pipe under the filter are three gates, 2, 4, and 5, as shown in Fig. 26. A 2-in. supply pipe from the city main, with a gate (3), and the 4-in. filter influent pipe, are connected to the supply pipe between Gates 2 and 4. The influent pipe has

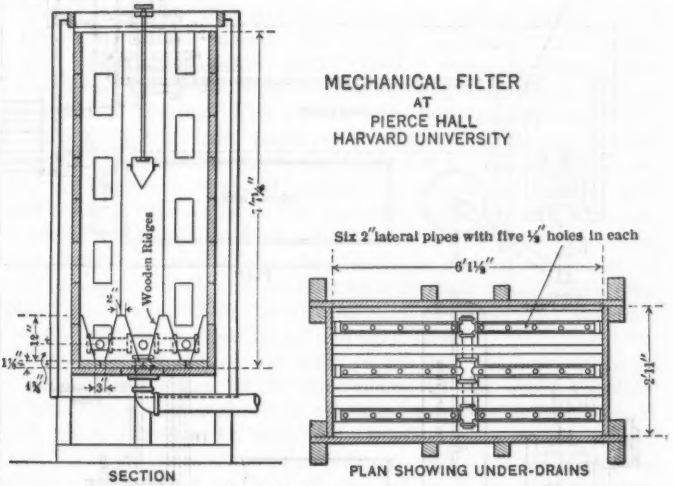

FIG. 25.

a gate (1) above the floor. Between Gates 4 and 5 there is a 4-in. pipe which connects with the effluent pipe between the under-drains and the effluent gates (6). This pipe may be used to carry the wash-water or air to the under-drains, or the effluent to the sewer, as Gate 5 connects the supply pipe with the city sewer. By this system of piping either reservoir or city water can be used for influent or washing purposes.

Four different filter beds were placed and tested. The first two were built above a roughly constructed system of under-drains, and the third and fourth over those shown in Fig. 25.

General Method.—To determine the effect of water-wash on the filter bed, water was forced into the filter through the under-drains at

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different rates, and was observed through windows in the sides of the filter box. High rates of wash were applied gradually and suddenly, both to clogged and unclogged beds. The effect of air-wash was ob-

ARRANGEMENT
OF PIPING AND APPARATUS FOR
EXPERIMENTAL FILTERS.
AT PIERCE HALL, HARVARD UNIVERSITY.

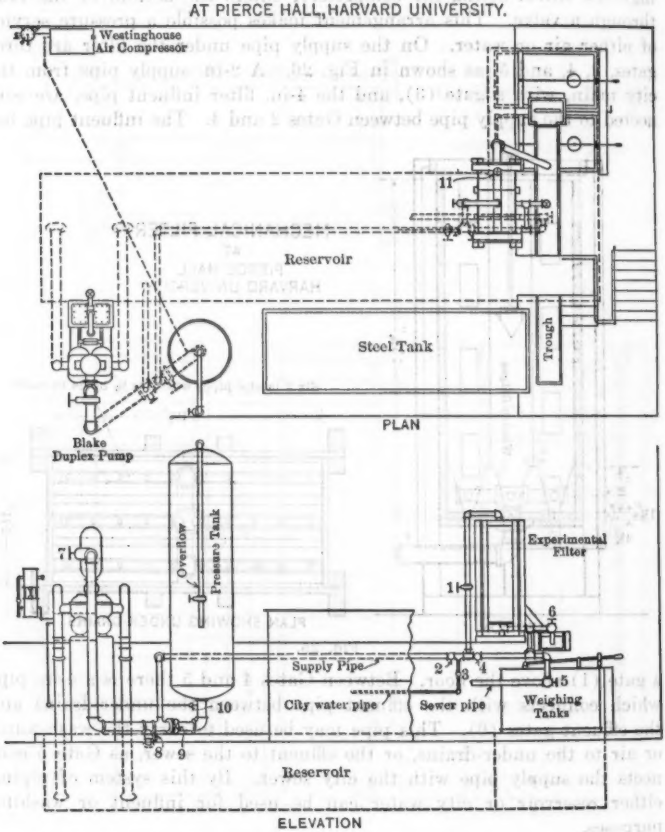

FIG. 26.

served by forcing air through the under-drains into the filter, with the bed clogged and unclogged. Samples of sand, taken at different depths below the surface, were subjected to mechanical analysis, in order to determine the grading effect of the wash.

The loss of head in washing was ascertained by noting the pressures in the under-drains and at different elevations in the filter bed. Mr. Pirnie.

The loss of head during filtration was ascertained by allowing alum-treated water to enter the filter above the sand, flow through the bed, and discharge from the under-drains at determined rates, noting periodically the total loss of head, and the loss of head between different elevations in the bed.

An idea of the intensity and depth of clogging in the sand was obtained from the turbidity of sand samples scraped from the surface and at different depths below the surface of clogged beds. Throughout all the tests watch was kept at the windows in the filter box to observe everything that happened.

First Bed.—In the first strainer system the central laterals were 1 in. lower than the side laterals, their center line being 3.75 in. above the floor. The ridges (of ¾-in. planks nailed to triangular braces) were so uneven that wash-water could enter under them and pass up through large cracks in the top. The laterals were 2-in. wrought-iron pipes, each having six ½-in. holes staggered ½ in. on each side of the lower center line. Over this system, to the depth of 6 in. above the floor, was placed the first grade of gravel, passing a 1.5-mesh and retained on a 1-mesh screen. Practically all this gravel remained above the laterals. The second grade of gravel passed a 1-mesh and was retained on a 3-mesh screen. It was 3.5 in. deep, and was covered with 1 in. of gravel passing a 3-mesh and retained on a 10-mesh screen. The total depth of gravel was 10.5 in. Next came 4 in. of coarse sand, and then 28 in. of a mixture of Plum Island and bank sand.

Second Bed.—A change was made in the first strainer system, all the laterals being placed in the same plane, 4.75 in. above the floor. The same ridges were used. Care was taken to pack the first grade of gravel, which passed a 1.5-mesh and remained on a 1-mesh screen, around five of the laterals, one being left unpacked. The top of the first grade was 5.5 in. above the floor; the second grade (passing a 1-mesh and retained on a 3-mesh screen) was 6.25 in. deep, and the fourth grade (passing a 3-mesh and retained on a 10-mesh screen) was 1.25 in. deep. The total depth of the gravel was 13 in. Over this was placed 3.25 in. of coarse sand and 30.25 in. of Plum Island sand, effective size 0.57 mm. and uniformity coefficient 1.70.

Third Bed.—The strainer system and the ridges previously described (Fig. 24) were placed for this bed, and the largest size of gravel used in the first two beds was discarded. The first grade of gravel (passing a 1-mesh and retained on a 2-mesh screen) was packed carefully about the laterals, and enough was used to cover the ridges, bringing the surface to 12.5 in. above the floor. The second grade (passing a 2-mesh and retained on a 3-mesh screen) was 3 in. deep; the third (passing a 3-mesh and retained on an 8-mesh screen) was

Mr. Pirnie. 2 in. deep; and the fourth (passing an 8-mesh and retained on a 10-mesh screen) was 1 in. deep, making the total depth of gravel 18.5 in. Above this was placed 21 in. of Plum Island sand, the effective size of which was 0.57 mm. and the uniformity coefficient 1.70.

Fourth Bed.—The sand of the third bed was removed, and 23¾ in. of white Ipswich beach sand was substituted, having an effective size of 0.37 mm., and a uniformity coefficient of 1.65. The gravel and underdrains were the same as those for the third bed.

Details of Experiments.

The loss of head was determined at different periods during the run by reading all the gauges, and the rate was ascertained by weighing the effluent. When the head became negative at any one gauge, the positive-head gauge was shut off, and the negative-head gauge at the same elevation in the bed was turned on. A careful watch was kept on the sand bed through the windows and on the air indicator. In removing the sand beds, care was taken not to disturb the conditions at the surface of the gravel, and, when within a few inches of the gravel, the sand was scraped off carefully so that its condition in the parts of the bed away from the windows could be seen.

In replacing the filter bed, each grade of gravel was carefully leveled before the next grade was placed, and, when the last grade had been smoothed off, the filter was filled with water up to the gutter. The sand was dumped into the water, and settled on the gravel in piles, after which the wash was turned on, causing the bed to come to a uniform level.

First Bed.—On the speaker's arrival in Cambridge, February 3d, 1913, the first bed had already been placed. It was in poor condition, as the sand was mixed with the gravel. When the filter was running, sand escaped continuously from the effluent pipe. The bed was in such poor condition that no quantitative tests were made, but a few preliminary observations were taken in order to test out various parts of the apparatus. The filter was run at the rate of 125 000 000 gal. per acre per day, with alum applied to the influent, and the bed was watched through the windows. After running several hours, conditions of negative head were established, and the sand began to dry out about 6 in. below the surface, the latter being very dirty and covered with a slimy coat of alum flock. The sand was discolored from 3 to 4 in., varying from a marked stain at the surface to no noticeable color 4 in. below. The drying out in the sand increased, and the rate of filtration decreased rapidly. On closing the effluent gate suddenly air began to escape on the surface of the bed, giving an appearance like the start of an air-wash.

In a clogged condition the bed was washed with water applied suddenly at a rate of 2.6 ft. per min. The imprisoned air escaped first

at several points near the sides of the box, and then from the surface of the sand. Then 6 in. of the bed rose in a mass, hit the bottom of the gutter, and broke. Marked streams of greatly agitated sand shot up through a bed of less agitated sand. These streams waved slightly in their upward course, but the points in the gravel where they originated were fixed, showing that the wash-water had a much greater velocity at these points than at others. Some of these streams were swifter than others, and when they reached the top of the sand it seemed to boil. The sand between the streams had a downward motion, and was caught at the gravel and shot upward in the currents. The velocity of wash was noticeably greater at the sides and ends of the box than in the middle. Mr. Pirnie.

After washing, the water was drawn from the filter and the surface inspected. About the ends and sides of the filter there were clearly marked spots of coarse sand which were 5 or 10 sq. in. in area and formed the tops of shafts of coarse sand extending vertically to the gravel. These shafts had been formed at places in the bed where there was higher velocity of wash-water. After experimenting with this bed long enough to obtain a clear idea of its imperfections, it was taken out.

Second Bed.—After the gravel and 3.25 in. of coarse sand had been put in place, a wash of 2.5 ft. per min. was turned on. The agitation seemed to be greater at the ends of the box than in the main part of the bed. The greatest agitation was over the lateral which had been left unpacked in setting up the bed. When all the sand was placed in the bed and the wash was turned on, a very pronounced stream of swiftly moving sand extended upward from the top of the gravel directly over the unpacked lateral. It was accompanied by a violent boiling at the surface. There were other streams which were not so marked, and between them the sand descended slowly, only to be caught and shot up again by the currents. The streams over the packed laterals seemed to be very uniform in upward velocity, and their courses shifted back and forth on the way to the surface. On the whole, the wash was more uniform than in the first bed, but did not appear to be uniform enough to insure against the mixing of sand with the gravel.

After the wash the filter was drained and the surface of the bed examined. The central part presented a smooth surface of fine sand, but, close to the sides, there were several small patches and one large one of coarse sand near the end over the unpacked lateral. These spots occurred wherever the velocity of the wash-water was greatest.

An explanation of what is meant by the term "negative head" will make its use clearer. A filter in operation has a given head of water above the sand bed and is discharging water through its under-drains at a given rate. As the water passes through the filter bed into the under-drains there is a loss of head due to eddies and frictional re-

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sistance which is equal to the difference between the head as registered on the top of the sand and that in the under-drains. Now, if the top few inches of the sand are clogged so that more head than that available above the surface is necessary to force water through at the given rate, the additional head is obtained from water below the clogged part of the bed, that is, the water below pulls on that above just enough to force it through the clogged portion at the given rate. In general, when the loss of head in passing the sand at a given point in the bed exceeds the head of water above that point, a suction exists equal to the difference between the total loss of head and the head available above that point. This suction is called the negative head at the point in question.

The dot and dash line cutting two of the curves (Fig. 27) gives for any point in the sand bed the total head of water above that point; hence, at any point, if the plotted loss of head falls to the right of this line the head is negative by the amount it falls to the right, and, if it falls to the left, the head is positive by the amount it falls to the left. As the loss of head in the lower part of the filter bed is constant for constant rates, and is only a fraction of the difference in elevation between the top and bottom of this part of the bed, some of this head may be used in pulling water through the top of the sand and still leave a positive head on the under-drains. Thus, a considerable negative head may exist just under the surface of the sand, though the head is still positive on the under-drains. Negative head in a filter is always at its maximum just under the clogged surface of the sand, and this is where air first begins to free itself from the passing water and fill the sand voids.

Water subjected to reduced pressure gives up air, which at first collects in little pockets in the sand voids. The surface coating of the sand, under the increasing pressure, breaks through in places from time to time, suddenly compressing an air pocket which breaks the surface in another place and escapes. Some flock is carried through the breaks, and forms another coating deeper in the bed. This action continues until the clogged portion of the sand is too strong to be broken, and then air accumulates in the sand voids until a cushion of air is formed between the clogged surface sand and the reservoir of water supplying the under-drains. As soon as this happens, the hydraulic unity of the filter is destroyed, and two separate actions begin, namely, the discharge of water above the sand through the clogged surface into rarified air, and the discharge from a body of water under a partial vacuum through the effluent-gate into water under atmospheric pressure. Air continues to escape from the water that passes through the surface of the sand, and, expanding, reduces the available negative head by replacing with air part of the head on the under-drains. The discharge of water through the clogged surface

decreases rapidly as the available negative head is reduced, and soon all the water in the lower part of the bed is discharged, leaving the bed filled with air and supporting above its clogged surface a reservoir of water. This rapid reduction in available negative head was due to the fact that the filter box was not absolutely tight, and air leaking in quickly reduced the vacuum when the head became negative in the vicinity of a leak. It is very important, therefore, to have a filter absolutely tight, if it is to be operated under negative head. The reduction of available negative head was much more rapid in the experimental filter than it would be in a tight concrete unit. Mr. Pirnie.

CURVES FOR LOSS OF HEAD TEST
FROM TABLE 5.
FEBRUARY 25TH, 1913.
Distribution of Loss at Different Periods during Run.

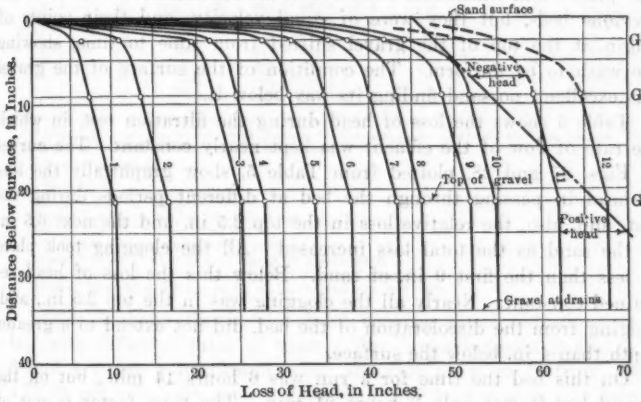

FIG. 27.

The curves showing the comparative losses of head in the top of the sand explain themselves. By far the greatest loss is in the top 1½ in. of sand, and the rest of the variable loss takes place in less than the next 11½ in. The loss below Gauge 3 remained constant throughout the run.

An air-wash was applied to the filter in this clogged condition. The surface was completely broken up, and the dirt and top sand shot up into the water standing a little below the level of the gutter. Water penetrated only 4 or 5 in. into the bed, as the rising air kept it out of the lower portion, and therefore there was no disturbance between the sand and gravel. At the end of the air-wash the water above the sand was black with flock, and the surface was in mounds and valleys. The application of a water-wash cleared away all the dirt, and at the end left the bed smooth and clean.

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Throughout the experiments with the second bed, no sand escaped from the effluent pipe, and, on taking out the bed, it was found to be in very good condition. In a few places, however, sand was mixed with the fine grade of gravel, and this was very markedly the case over the unpacked lateral near the end of the filter.

Third Bed.—After the gravel had been put in place, a wash of 2.5 ft. per min. was turned on. The water was clear enough to allow inspection of the surface of the fine gravel for a considerable distance back from the windows, and it was seen to be very uniformly agitated. There was no fixed point of boiling, thus showing the velocity of wash to be uniform throughout the main part of the bed. The agitation at the ends, however, appeared to be slightly greater. The sand was placed and washed at the same rate. Streams existed, as in the previous beds, but they were of equal velocity, and their points of origin at the top of the gravel shifted from time to time, showing the wash to be uniform. The condition of the surface of the gravel was excellent, no sand finding its way below it.

Table 5 shows the loss of head during the filtration test, in which the rate of flow of the effluent was kept nearly constant. The curves of Figs. 27 and 28, plotted from Table 5, show graphically the loss of head in passing through the bed at different periods during the test, and, also, the relative loss in the top 2.5 in. and the next 6.5 in. of the sand as the total loss increased. All the clogging took place in less than the first 9 in. of sand. Below this the loss of head remained constant. Nearly all the clogging was in the top 2.5 in., and, judging from the discoloration of the bed, did not extend to a greater depth than 4 in. below the surface.

On this bed the time for a run was 6 hours 14 min., but on the second bed it was only 3 hours 21 min. The time factor is not of much use, as an unknown quantity of alum was applied to the influent, which varied in alkalinity; but it is obvious that, the longer the occurrence of negative head can be postponed, the longer the filter will run, and this was accomplished in the tests on the third bed by having a greater head of water above the sand than was available on the second bed. Near the end of the test the same conditions existed as in the tests on the second bed.

A sudden high rate of wash was turned on the bed after it was thoroughly clogged. The imprisoned air was compressed until it lifted and broke the surface, causing conditions similar to the start of an air-wash. Water rose through the gravel, lifted the sand in places, and then produced the condition of streams, previously described. No mixing of the sand and gravel was caused by the sudden application of the wash.

TABLE 5.—LOSS OF HEAD TEST. Mr. Pirnie.

February 25th, 1913.

No.	Time: Hrs.	Time: Min.	Loss of Head, in Inches, Between Gauges: 1 and 2.	2 and 3.	3 and 4.	4 and 6.	Loss of Head, in Inches per Inch, Between Gauges: 1 and 2.	2 and 3.	3 and 4.	4 and 6.	Total loss of head, in inches.	Rate, in millions of gallons per acre, per day.
1	10	42	6¼	1	3¼	2½	2.5	0.2	Const.	Const.	13	120
2	11	7	11	2½	3¼	2¾	4.4	0.4	.....	.....	19¼	123
3	12	4	19¼	2½	3¼	2½	7.8	0.4	.....	.....	27¾	122
4	12	23	21⅞	2¾	3	2½	8.8	0.4	.....	.....	30⅝	123
5	1	8	27½	3¼	3	2½	11.0	0.5	.....	.....	36¼	126
6	1	36	31	3⅞	3	2¾	12.4	0.6	.....	.....	40⅝	126
7	2	16	35¼	4¾	3¼	2¾	14.1	0.7	.....	.....	46	129
8	2	41	37½	5⅛	3⅛	2⅞	15.0	0.8	.....	.....	49	129
9	3	21	41¾	6¾	3¼	2¾	16.7	1.0	.....	.....	54¼	126
10	3	45	44	7¾	3	2¾	17.6	1.2	.....	.....	57½	126
11	4	5	59		3¼	2¾	.....	.....	.....	.....	65	122†
12	4	16	64⅝		3¼	2¾	.....	.....	.....	.....	70⅝	122
13	4	34	71		3¼	2½	.....	.....	.....	.....	76¾	122‡
14	4	46	74¼		3½	2½	.....	.....	.....	.....	80¼	122
15	4	56	76½		7¼*	2¾	.....	.....	.....	.....	86½	122

*There was no negative-head gauge at Gauge 2, so that when this gauge became negative the losses between 1 and 3 were read.

†2 and 3 negative.

‡4 negative.

Gauge 5 was out of order.
Gauge 2 is 2¼ in. under sand surface.
Gauge 3 is 9 in. under sand surface.
Gauge 4 is 21¼ in. under sand surface.
Gauge 6 connected with effluent pipe above gate.
Gauge 4 is ⅝ in. below top of fourth grade of gravel.
Loss of head in gravel = ¼ in. at 125 000 000 gal. per day.

Table 6 shows the loss of head between given elevations in the bed with different rates of wash. The losses between 2 and 3 and 3 and 4 reach a maximum when the rate of wash is from 1.43 to 1.77 ft. per min. At rates higher than 1.77 ft. per min., the losses of head decreased. Between Gauges 4 and 5 in the gravel, the loss of head increases steadily with the increase in rate of wash, which is true also between Gauges 5 and 6, as the water passes from the under-drains into the gravel. The perforations in the under-drains, as well as the voids in the gravel, remain constant during the wash. Therefore, as the velocity of wash increases, the losses of head in the under-drains and the gravel increase. It is different with the sand, however. The upward current of water exerts a frictional force on a sand grain, its intensity depending on the velocity of the water and the size and shape of the grain, and when this force exceeds the weight of the grain in water, it is carried up until a position is reached where the velocity, and therefore the frictional force, is less. Thus the finest grains of sand find their way to the top and become suspended in the rising column of wash-water just near enough together to cause the velocity required to keep them in their suspended positions.

Mr. Pirnie. As the rate of wash is increased, heavier particles become suspended below the lighter ones, and the total voids in the sand are correspondingly increased. A measurement of the increase in voids may be obtained by multiplying the area of the filter by the distance the sand surface is raised. When the sand particles are suspended, there can be no greater scrubbing action caused by a higher rate of wash. The voids simply increase in size, allowing the velocity of the water passing the sand grains to remain the same. Therefore, for this sand, the maximum useful wash rate would seem to be 1.77 ft. per min. Greater rates increase the voids and do not increase the scrubbing action on the particles.

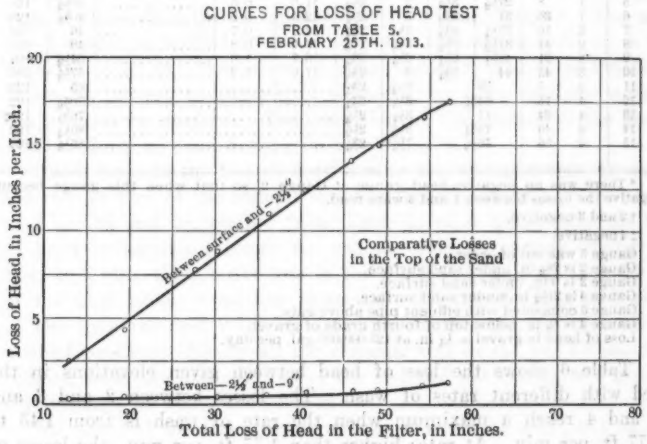

FIG. 28.

At the end of the test, little lumps of sand, held together by some binding material, settled back on the surface. These were not present after an air-wash, but they are not broken up by a water-wash. They are suspended out of reach of the stirring action, and settle back unbroken on the surface after the wash. These masses reduce the clear surface of the bed. This may be desirable, inasmuch as it helps to form more quickly the desired filtering surface. If it is undesirable, the best way to break up these masses is with an air-wash.

Fig. 29 shows the grading of the sand in the third bed, due to washing. It is interesting as a proof of the way the sand is suspended by the upward current of water. On turning off the wash, the sand settles according to its size, the larger particles settling out first, and the smaller ones being deposited in successive layers

on the coarser ones below them. The wash tends to make a mechanical separation of the entire bed of sand, and, because the sand curve is a straight line on a multiple logarithmic scale, in the natural sand, it is graded in the same way in a mechanical filter. The uniformity coefficient approaches unity at the surface, but, owing to foreign matter and irregularly shaped particles of sand, it is not unity. Deeper in the bed the stirring action of the streams mixes some larger particles with the sand, so that the uniformity coefficient is increased. Mr. Pirnie.

TABLE 6.—Loss of Head in Washing.

March 8th, 1913.

Rate, in feet per minute.	Distance sand surface raised, in inches.	Gauge Readings, in Inches.						Pressure gauge 7 lb. per square inch.	Loss of Head Between Gauges, in Inches.					Head outside under-drains 84.5 Gauge 5.	Head inside under-drains 84.5 Gauge 6.	Resultant head, in inches.
		1	2	3	4	5	6 computed from 1.		1 and 2.	2 and 3.	3 and 4.	4 and 5.	5 and 6.			
0.30	.....	33½	31¾	27¾	21⅝	21½	18½	2.25	1¾	4	6⅛	⅛	3	63	66	3
0.76	.....	33⅛	30⅝	25⅜	16¼	15¾	— 5½	3.25	2½	5	9⅜	½	21¼	68¾	90	21
1.15	.....	33	30¼	25¼	15½	14¼	— 32	4.20	2¾	5	9¾	1¼	46	70¼	116	46
1.43	⅝	32⅞	30⅛	24⅞	14¾	13⅛	— 50	4.84	2¾	5¼	10⅛	1⅝	63	71⅝	134	63
1.77	.....	32⅝	29⅝	24¼	14⅛	12½	— 96	6.50	3	5⅝	10⅛	1⅝	108	72	180	108
2.05	1⅞	31⅞	28½	23¼	13¼	11	— 134	7.90	3⅜	5¼	10	2¼	145	73¼	218	145
2.42	3	28⅛	25	20⅛	10½	6¾	— 178	9.50	3⅛	4⅞	9⅝	3¾	185	77¾	262	195
2.76	.....	24½	21	16	6	2	— 240	11.70	3½	5	10	4	242	82½	324	242

Pressure gauge has zero correction of + 1.5 lb. per sq. in., which has been applied in this table.
Gauge reading at under-drains = 84.5 in.
Gauge 5 is in gravel at drains 1¾ in. above center line.
Gauge 4 is in gravel 5⅝ in. below top.
Gauge 3 is in sand 11⅝ in. above gravel.
Gauge 2 is in sand 18½ in. above gravel.
Gauge 1 is above sand.

Fig. 30 gives a good indication of the depth and intensity of clogging in the third bed. From inspection of the bed after clogging it was apparent that the surface sand was discolored to a depth of 4 in., varying from a very marked discoloration at the surface to no noticeable color deeper than 4 in. below the surface. Comparing the turbidity curve of samples scraped from a washed bed with that curve for samples scraped from a clogged bed, it can be seen that the dirt does not penetrate below the upper 5 or 6 in. of the bed. This method does not give an accurate record of the intensity of clogging at different depths. Some of the dirt was in lumps, or adhered to the sand particles after drying, and therefore did not add to the turbidity of the water with which the sample was shaken. The curves, however, indicate clearly the way in which the clogging was distributed.

An air-wash was applied to the bed in an unclogged condition. At the start the bed was full of water, and the rising bubbles thoroughly mixed the sand and finest gravel. Almost instantly, however, the

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water was driven out of the fine gravel and sand by the rising air, causing all disturbances to cease except those at the surface of the sand. At the start the air pressure in the under-drains was 2 lb. per sq. in., and this was reduced to 1.5 lb. per sq. in. when the water was forced out of the sand. As the water stood at the level of the gutter when the wash was started, an overflow occurred, and carried with it a considerable quantity of sand. This was thoroughly mixed with the water, owing to the violent stirring action of the air bubbles. The overflow ceased as soon as the water had been expelled from the bed. This waste of sand could be avoided by drawing the water below the level of the gutter before starting an air-wash on an unclogged bed. It would not be necessary on a thoroughly clogged bed filled with air.

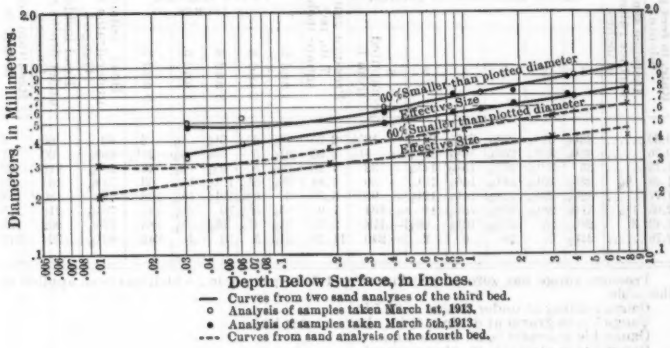


FIG. 29.

After the air-wash, a high-velocity water-wash was applied, driving the sand out of the fine gravel and leaving the bed in as good condition as it was before the test. The little masses of sand which existed after a water-wash alone were not present after this test, showing that an air-wash preceding a water-wash is the more thorough method of cleaning the sand.

When the sand of the third bed was removed, the gravel everywhere was found to be in the same condition as when placed. No sand was mixed with it. In view of the fact that all possible variations of washing were used without producing permanent bad effects, this form of bed was considered satisfactory.

Fourth Bed.—When the Ipswich Beach sand had been placed in the filter, a water-wash was applied to grade it out. The desired effect was produced in this bed with a considerably lower rate of wash than was necessary to produce the same effect on the third bed. A few shells and some gravel separated from the sand, and formed on top of

the gravel originally in the filter. The sand was graded according to size above it. Mr. Pirnie.

All the clogging took place in less than the upper 5.25 in. and most of it took place in the upper 0.5 in. Negative head occurred in the same way, and with the same effect as in the other three beds. The run was of much shorter duration, owing to the fineness of the sand used. The discoloration in the sand extended from $\frac{1}{2}$ to $\frac{3}{4}$ in. below the surface, giving a visible measurement of the depth of clogging. This was about one-sixth as great as the depth of clogging in the third bed.

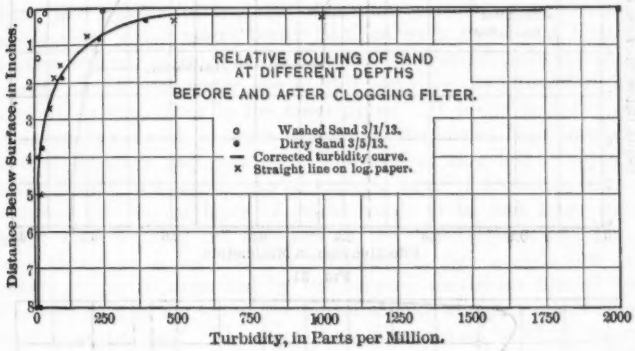


FIG. 30.

The top 8 in. of sand was suspended with a wash rate of 1.15 ft. per min., a condition which was not produced on the third bed until a wash rate of more than 2.5 ft. per min. was used. As described under the third bed, greater rates of wash than that necessary to suspend the sand produced little additional cleansing action. Therefore, for the sand used in the fourth bed a maximum rate of 1 ft. per min. would seem to be more than ample.

A high-velocity wash applied suddenly to this bed lifted it bodily from the gravel before it broke up and began the stream action in the sand. No sudden washes with air or water seemed to affect seriously the condition of the boundary between the sand and gravel. The air-wash applied to the unclogged bed produced the most serious results. These were apparently remedied by the application of the high-velocity water-wash.

Fig. 29 demonstrates the grading of sand in the fourth bed due to washing. By comparing the curves a good idea may be had of the relative sizes of the sand in the third and fourth beds.

Figs. 31 and 32 show the effect of washing on sand placed in a mechanical filter. When the sand is first placed in the filter the effec-

Mr. Pirnie.

tive size and uniformity coefficient are the same at all elevations in the bed. The wash causes a mechanical separation of the different sizes. The finest grains are forced to the surface, and the larger ones are adjusted in layers below the surface according to size.

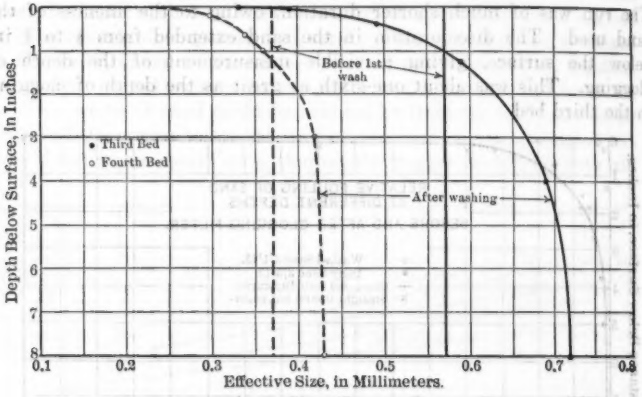


FIG. 31.

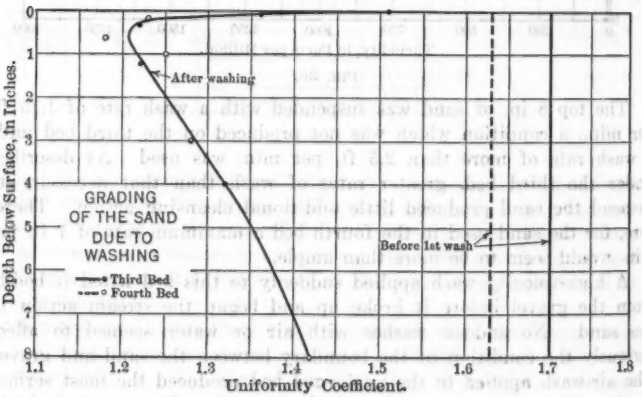


FIG. 32.

Conclusions.—If a filter can be constructed so that the wash-water has a uniform velocity when it passes from the gravel into the sand, there will be no tendency of the gravel and sand to mix. This condition was so nearly attained in the third and fourth beds that the sand remained above the gravel after very high rates of wash had been suddenly applied and shut off. The tendency toward a slightly

greater velocity of wash at the ends of the filter than in the main part of the bed was probably caused largely by the smoothness of the walls. The gravel between the ridges conducted the water with little loss of head to the ends, where there was less resistance afforded by the sand against the smooth filter walls than by the sand in other portions of the bed. There is no reason for building ridges parallel to the under-drains and not perpendicular to them. The tendency toward unequal velocities in one direction is as great as in another. Thus, if ridges are to serve their purpose completely, they should be constructed in the form of concentric rectangles about the center of the filter. The speaker believes that ridges are worse than useless. They occupy space which might better be filled with gravel and thus give a larger reservoir of voids at the bottom. This would make the upward velocity of the wash-water less at this elevation, and therefore allow a greater baffling effect in the lower gravel. It would be better, also, to use smaller gravel, in order to increase the loss of head through it, both in filtration and washing. The greater this loss of head the nearer are the ideal conditions of uniform rates of filtration and wash realized. The lowest layer of gravel needs to be just large enough not to enter or plug the openings in the under-drains, and the top layer should be a considerable thickness of fine gravel, just large enough not to be disturbed by the adopted maximum rate of wash. The sand next to the top of the gravel should be only slightly smaller than the gravel itself. The sides of the filter should be rough, in order to increase the friction and remove the possibility of unequal velocities of wash. Mr. Pirnie.

The strainer system, Fig. 25, is excellent, and, if covered with from 18 to 24 in. of gravel ranging in sizes from coarse at the bottom to fine at the top, should produce a uniform velocity of wash through the sand.

Water-wash, when the rate is high enough to suspend the sand grains, grades the sand according to size. The finest sand is on the top, and the successive lower layers are each coarser than those above.

The wash tests on the third and fourth beds indicate that, for sands of different effective sizes, there are different effective rates of wash. Higher rates give little or no additional scrubbing effect, but may be high enough to carry away the lumps which are formed in some filters.

Air-wash is necessary to break up lumps that form on the surface of the sand. A very high-velocity water-wash will not do this, although, if high enough, it will get rid of some of the lumps by washing them into the gutters. If the sand and gravel are filled with air before an air-wash is applied, no disturbance is caused, except in the top few inches of the sand, and this produces no mixing of the sand and gravel. If, however, the bed is filled with water at the start of the air-wash, the rising bubbles cause violent eddies which mix the fine

Mr. Pirnie.

gravel and sand to an alarming extent before the water is driven out. The high-velocity water-wash regrades the sand and fine gravel, and therefore no permanent injury is done by the air-wash; but it is necessary to have a 3 or 4-in. thickness of fine gravel to make sure that no sand reaches the coarser gravel, and it is also necessary to follow an air-wash with a high-velocity water-wash to expel from the gravel the sand that has penetrated into it. A low-velocity water-wash will not force the sand out of the gravel, and, if an air-wash is continuously used on a filter, followed by a low-velocity water-wash, this sand may work its way down to the under-drains.

During filtration, all the loss of head due to clogging takes place in the top few inches of the sand. Therefore, negative head first occurs just under the clogged surface, and may exist there while the head is still positive on the under-drains. The maximum negative head is always just under the clogged surface of the sand. Therefore, air first begins to be freed from the water and to collect there. The water entering the under-drains has previously been subjected to a higher vacuum than can exist at the under-drains, and therefore should not release air when passing into them. Air that forms under the clogged surface reduces the available negative head by reducing the depth of the reservoir of water supplying the under-drains. A very small leak in the filter walls will supply air in larger quantities under the clogged surface, and therefore reduce the available negative head more rapidly. There is also an element of danger in operating under negative head if air leaks occur below the surface of the gravel. A procession of bubbles will begin to rise through the gravel and sand, collecting under the clogged surface, as soon as the head becomes negative at the leak. This will cause the same mixing between the gravel and sand as that which takes place at the start of an air-wash, and the mixing will continue. The water passing downward in the process of filtration will help to carry the sand down into the coarser gravel, where it will remain when the filter is washed. Each time, toward the end of a run, the disturbance will occur at the same place, and, in this way, a considerable quantity of sand may pass through the gravel to the under-drains. Therefore, it is much better to increase the head above the sand than to try to operate under a large negative head.

The depth of the sand is an important consideration in the design of mechanical filters. Every unnecessary inch of depth means the sacrifice of positive head equal to 1 in. plus the loss of head in passing through 1 in. of the unclogged portion of the sand. If a very uniform rate of filtration can be realized, it may be that the depth of sand can safely be reduced. Filtration at such a high rate is almost, if not entirely, a straining process. Therefore, very little benefit could

be expected from longer contact with sand after the water has passed through the finest straining layers. In the coarsest sand used in these tests the depth of clogging was only 6 in. This indicates that a depth of sand of from 12 to 18 in. would be ample. Mr. Pirnie.

The results of the experiments left no doubt of the desirability of producing a uniform rate of wash over the entire bed. It is obvious that this may be brought about in much less space by directing the wash-water downward as it leaves the under-drains, thus breaking up the force of the individual streams against the solid filter floor. Sufficient gravel may be placed over the drains to quiet the eddies and currents remaining after the force of the streams has been broken. This depth may be much less than would be necessary if the wash-water streams were directed upward. In this case the gravel must break up the velocity of the streams as well as still the eddies and currents caused by them.

JOHN H. GREGORY,* M. AM. SOC. C. E.—The authors, as well as several of those who have discussed the paper, have presented data of especial value to engineers and others having to do with the design, construction, and operation of rapid sand filter plants. Had this information been available 15 years ago, when the present type of rapid sand filter was first being worked out, the task of the engineer engaged in designing such filters would have been greatly simplified. Mr. Gregory.

With reference to the head required to wash filters, it should be noted that the experiments record the head necessary to pass the wash-water up through clean sand. The paper and discussion would be of still greater value if data could also be included showing what increased head, if any, is required to wash dirty filter sand, that is, sand in which the interstices have become clogged with dirt and other foreign matter in the actual process of filtration.

F. A. BARBOUR,† M. AM. SOC. C. E.—This paper is of particular value to those interested in the design and operation of rapid sand filters. It makes available information of which but little has previously been made public, and which will prove valuable in future work, if used with due regard to the relation between the experimental conditions on which the data are based and those of practical operation. Mr. Barbour.

In connection with the building of the filter plant at Akron, Ohio, an investigation of the effects of high-rate filter washes, similar to that described in the paper, was carried out, at the suggestion of the writer, by the New York Continental Jewell Filtration Company—the contractor on that work.

In the Akron filters, the Wheeler filter bottom was used, the right being granted, without payment of royalty by William Wheeler, M.

* New York City.
† Boston, Mass.

Mr. Barbour.

Am. Soc. C. E., the designer and patentee. This filter floor consists of pyramidal depressions, 5.5 in. deep and 8.8 in. across the base, with a $\frac{3}{4}$-in. brass tube, 1.49 in. long, set at the apex, for the discharge of the filtered water or the admission of wash-water from the lateral channels or collectors below. The dimensions of the pyramidal depressions are such that five cement balls, 3 in. in diameter, one at the center at the apex of the $\frac{3}{4}$-in. opening, and four others in an upper layer, are in contact with each other and with the sides of the inverted pyramid. In the spaces between the upper four balls, nine marbles, each $1\frac{1}{4}$ in. in diameter, are laid, and on this a 6-in. layer of gravel is placed. Two views of this filter floor are presented, Fig. 33 showing several of the depressions, one empty and the others with one or more of the balls in place, and Fig. 34 showing a portion of the completed floor in one of the Akron filters.

The type of filter bottom just described has merit in low first cost, in freedom from dead spaces, and in the absence of metal parts subject to depreciation; but the most interesting feature is the complete distribution of high wash-water flows by the balls and marbles, so that with rates of wash up to and exceeding 4 ft. per min., there is no perceptible movement of a 6-in. gravel layer, even before the sand is placed, and therefore no need of any wire cloth, such as was used originally in the plant at Cincinnati.

A small filter with a floor of this type had been constructed at Belfast, Me., previous to the Akron work, and R. S. Weston, M. Am. Soc. C. E., had demonstrated the possibilities of high-rate washes in an experimental filter. The contractor for the filter equipment at Akron, however, had no previous knowledge of this type of floor, and, at the suggestion of the writer, a small experimental filter, 2 ft. square, with a standard Wheeler floor of four depressions, overlaid by 6 in. of gravel and 30 in. of sand, with plate glass on two sides, was constructed. This was used in various studies of the loss of head, at different rates of wash, of the gradation in the sand grains resulting from high-rate washes, and of the relation between the rates of wash and the expansion and rise of sand.

The gravel consisted of three layers, the lowest $1\frac{1}{2}$ in. thick, of material passing a $\frac{3}{4}$-in. and held on $\frac{1}{2}$-in. mesh; the intermediate, $1\frac{1}{2}$ in. thick, of material passing a $\frac{1}{2}$-in. and held on a $\frac{3}{16}$-in. screen; and the top 3 in. of material passing a $\frac{3}{16}$-in. screen and held by a screen with 12 meshes per inch. The sand had an effective size of 0.40 mm. and a coefficient of uniformity of 1.60. The loss of head was determined by glass water columns.

Four curves are presented on Fig. 35, which show the loss of head under four conditions: through the $\frac{3}{4}$-in. opening and pyramidal depression without the balls in place, with the center ball in place, and

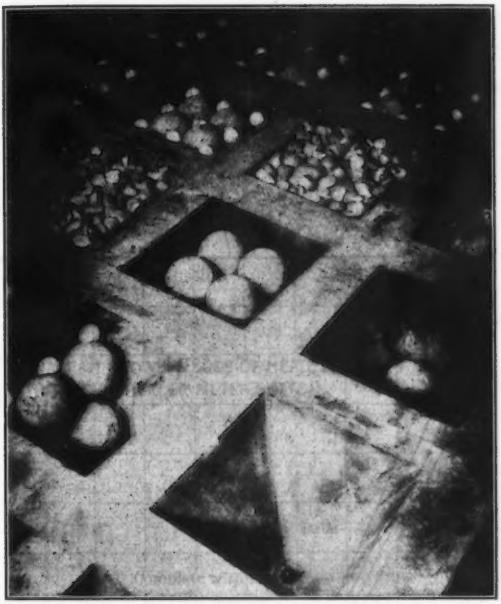


FIG. 33.—NEAR VIEW OF FILTER BOTTOM USED AT AKRON, OHIO, SHOWING METHOD OF PLACING BALLS AND MARBLES IN DEPRESSIONS.

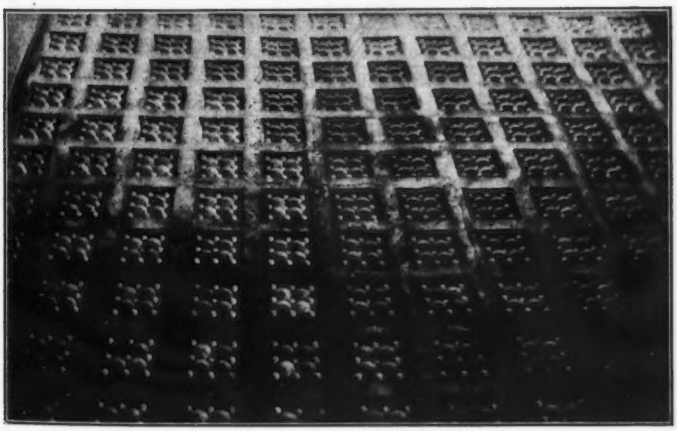
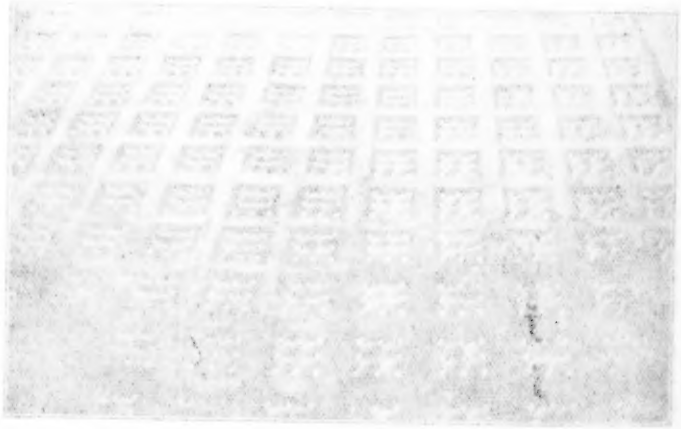


FIG. 34.—VIEW OF PORTION OF FILTER BOTTOM, AKRON, OHIO.



Mr. Barbour.

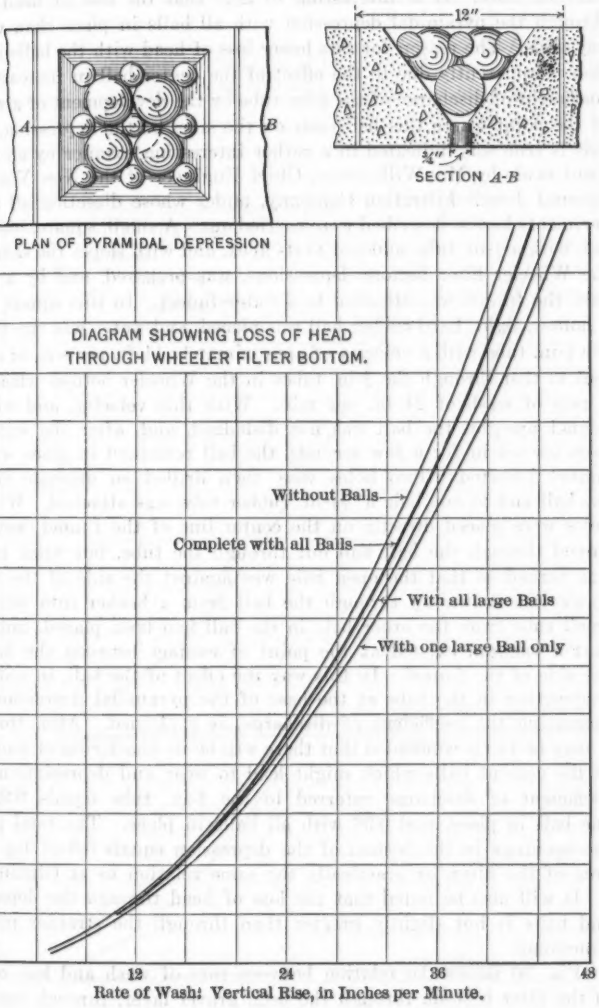


FIG. 35.

Mr. Barbour.

with the five 3-in. balls in place, and the five 3-in. and nine 1¼-in. balls in place. It is interesting to note that the loss of head is less through the pyramidal depression with all balls in place than without any balls. The reason for this lesser loss of head with the balls in position is apparently due to the effect of the center ball in increasing the coefficient of discharge of the ¾-in. tube by the development of a vacuum at the point where this ball rests on the sides of the depression. That this is true was indicated in a rather interesting manner by an experiment made by Mr. Williamson, Chief Engineer of the New York Continental Jewell Filtration Company, under whose direction all the experiments herein described were carried out. A small, square, brass funnel, with a ¼-in. tube soldered to its apex, and with slopes the same as in the Wheeler filter bottom depressions, was prepared, and by a rubber tube the funnel was attached to a water faucet. In this square funnel a hollow, light, hard-rubber ball was placed, and water was run through the ¼-in. tube with a velocity of approximately 11 ft. per sec., or equivalent to that through the ¾-in. tubes in the Wheeler bottom when using a rate of wash of 24 in. per min. With this velocity, and with the funnel upright, the ball was not dislodged, and, after the water had been turned in for a few seconds, the ball remained in place with the funnel inverted. Two holes were then drilled on opposite sides of the ball and to one hole a $\frac{1}{16}$-in. rubber tube was attached. When the holes were placed directly on the center line of the funnel, water was forced through the ball and out through the tube, but when the ball was turned so that the open hole was against the side of the funnel, water was drawn up through the ball from a beaker into which the small tube from the other hole in the ball had been placed, indicating that a vacuum existed at the point of contact between the ball and the side of the funnel. In this way the effect of the ball, in nullifying contraction in the tube at the base of the pyramidal depression, thus increasing the coefficient of discharge, is explained. Also, from this it may be fairly concluded that there will be no chattering or movement of the cement balls which might lead to wear and depreciation. The coefficient of discharge referred to the ¾-in. tube equals 0.92 with one ball in place, and 0.90 with all balls in place. The total area of the openings in the bottom of the depression equals 0.3 of 1% of the area of the filter, or practically the same relation as at Cincinnati.

It will also be noted that the loss of head through the depressions and balls is but slightly greater than through the strainer plates at Cincinnati.

Fig. 36 shows the relation between rate of wash and loss of head in the filter bottom, through the 6-in. gravel layer, through the 30-in. layer of sand, and through the complete filter. The rate of wash in the experiments ranged from 10 to 42 in. per min. It will be noted

Mr. Barbour.

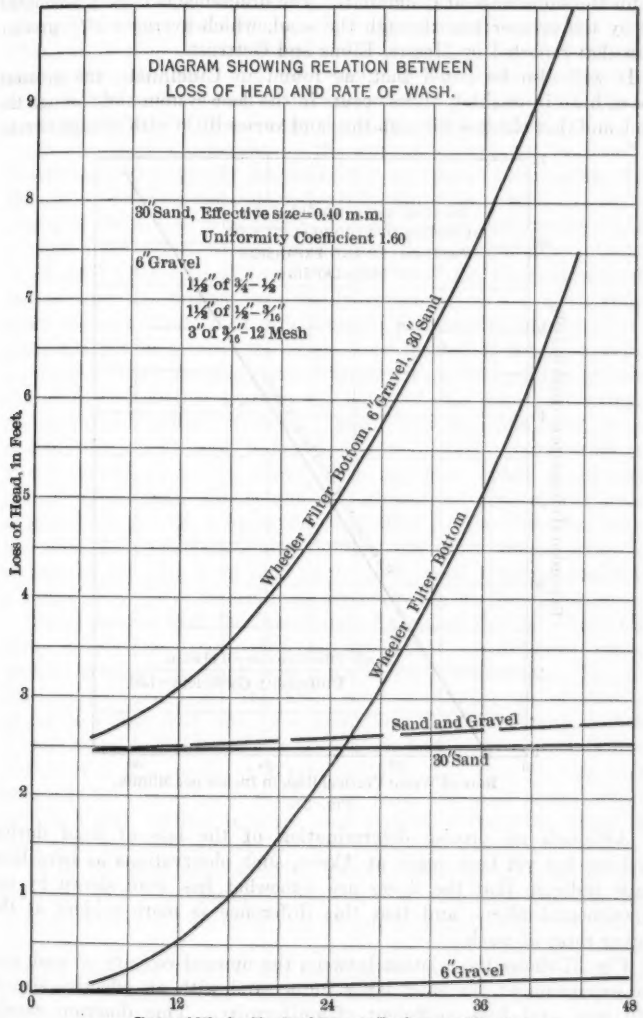


FIG. 36.

Mr. Barbour.

that the total loss for a 24-in. wash is 4.86 ft., as compared with 4.20 ft. for the same wash at Cincinnati. The difference is largely accounted for by the greater loss through the sand, which averages 25% greater than that reported by Messrs. Ellms and Gettrust.

It will also be noted that, as found in Cincinnati, the greatest loss of head in washing filters occurs in the floor system and through the sand, and that the loss through the sand varies little with change of rate.

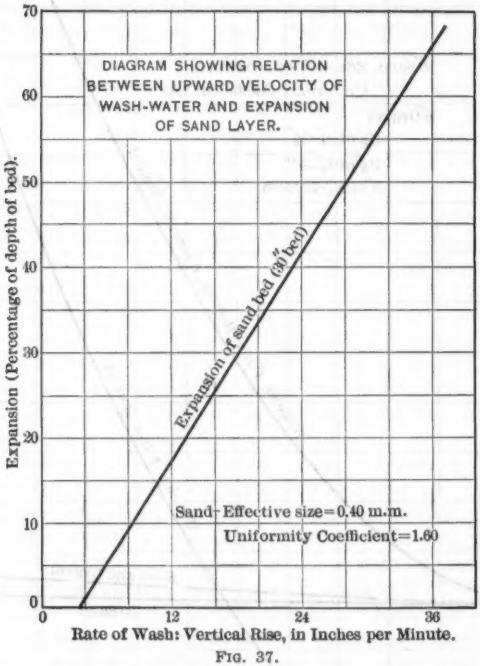


FIG. 37.

Although no precise determination of the loss of head during washing has yet been made at Akron, such observations as have been made indicate that the losses are somewhat less than shown by the experimental filters, and that this difference is more evident at the higher rates of wash.

Fig. 37 shows the relation between the upward velocity of wash and the expansion of the sand layer for a sand with an effective size of 0.40 mm. and 1.60 coefficient of uniformity. This diagram closely checks the curve of expansion for the sand with an effective size of 0.41 mm. shown on Fig. 19.

Mr. Barbour.

As pointed out by the authors, it is very necessary, in determining the height at which the gutters should be placed above the sand, to take into account the reduction in horizontal section by the gutters. Thus, at Akron, the horizontal area of the gutters is 30% of the total area of the filter, and the velocity of the wash-water equivalent to 24 in. per min., based on the area below the gutters, is equal to a velocity of 31.2 in. per min. at the plane of the gutters.

In any case, the height of the gutters should not be based too closely on experimentally determined expansions of clean sand. When the Akron filters were put in service, the sand, which as delivered averaged about 0.40 mm., with a coefficient of uniformity of 1.50, was given ten 5-min. washes at the rate of 24 in. per min. About 1½ in. of sand were scraped from the surface of the filters during these washes, and the surface was finally composed of a sand having an effective size of from 0.30 to 0.35 mm. For several months after the plant was put in service, the filters were washed at the rate of 24 in. per min., without apparent loss of sand, but as the sand grains became coated with vegetable matter, the color of the water averaging 45, the expansion gradually increased, and it has been necessary, in order to avoid the loss of sand, to lower the rate of wash progressively until, at the present time, it is about 17 in. per min. This would suggest that, in determining the height of gutters from experimental results, a leeway should be allowed for the effect of the filtering material becoming dirty, and therefore lighter. In other words, data based on experiments with clean sand provide a basis of judgment rather than a means of precisely determining the necessary height of gutter.

In connection with the furnishing of the sand for the Akron filters, some interesting experiments on the effect of different rates of wash in grading the sand were made by Mr. Williamson. The specifications required that "the filter sand shall have an effective size of not less than 0.35 mm., nor more than 0.44 mm., and shall have a uniformity coefficient of not more than 1.65." After specifying ten washes of 5 min. each, at a rate of 24 in. per min., it was further required that "samples of sand taken from the sand layers after the sand had been washed in place, and especially from the upper inch, shall meet the requirements for size heretofore specified."

With the gradation resulting from a wash at the rate of 24 in. per min., this specification is equivalent to requiring that the upper inch shall have an effective size of not less than 0.35 mm., and the lower inch an effective size not greater than 0.44 mm. To determine whether the sand which it was proposed to use at Akron would fulfill these requirements after washing, the experimental filter already described was filled to a depth of 30 in., and washed at various rates, samples being taken at different depths by thin, brass cylinders inserted through holes drilled in the sides of the filter.

Mr. Barbour.

The sand, as delivered, had an effective size of 0.34 mm. and 1.82 coefficient of uniformity. After washing for 6 min. at a rate of 16.6 in. per min., the original 30 in. was increased to 32.2 in., and a sample from the top inch had an effective size of 0.17 mm. and a 1.41 coefficient of uniformity. Then 2 in. were scraped from the surface, and the filter was again washed at a rate of 17.7 in. per min. after which seven samples were taken at the depths shown in Part A of Table 7.

TABLE 7.

Sample No.	Depth below surface, in inches.	Effective size, in millimeters.	Coefficient of uniformity.
PART A.			
3	0 - 1/2	0.24	1.25
4	1/2 - 1	0.32	1.22
5	1 - 2	0.34	1.23
6	2 - 3	0.35	1.23
7	3 - 4	0.36	1.28
8	4 - 5	0.40	1.40
9	7 - 8	0.45	1.40
PART B.			
14	1	0.30	1.27
15	7 1/4	0.45	1.27
16	13 1/4	0.48	1.37
17	19 1/4	0.48	1.37
18	24 1/4	0.48	1.37
19	30 3/4	0.48	1.37
PART C.			
21	1	0.35	1.51
22	6	0.42	1.45
23	12	0.42	1.45
24	18	0.42	1.45
25	23	0.42	1.45
26	29 1/2	0.44	1.45
PART D.			
27	1	0.41	1.46
28	5 3/4	0.38	1.53
29	11 3/4	0.38	1.50
30	17 3/4	0.41	1.46
31	22 3/4	0.44	1.43
32	29 1/4	0.43	1.49

After taking Sample No. 9, the filter was washed several times at rates varying from 13 to 25 in. per min., and 1/2 in. was scraped from the surface, making a total of 2 1/2 in. removed by scraping. Then 2 1/4 in. of the original sand were added, and the filter was washed four times at a rate of 26 in. per min., 1 in. of the surface sand being

removed in two scrapings of ¾ and ½ in. after the second and fourth washes, respectively. The filter was then given two 5-min. washings at 26 in. per min., and Samples Nos. 14 to 19 (Part B), Table 7, were collected. Mr. Barbour.

The filter was then given two washes at 13.1 in. per min., and Samples Nos. 21 to 26 (Part C), Table 7, were collected.

Again, the sand was given a 10-min. wash at the rate of 10.5 in. per min., and Samples Nos. 27 to 32 (Part D), Table 7, were collected.

A comparison of the results noted in Table 7 makes it evident that the higher the rate, the greater the difference between the effective size of the top and bottom layers, and the lower the coefficient of uniformity of each layer; or, in other words, the higher the rate, the more nearly is the sand graded into layers in which the particles are of the same size. Also, it is shown that, with the sand used, gradation did not result from a wash as low as 10.5 in. per min.

Gradation of the sand is immediately indicated by an increase in volume, and, when a low-rate wash is applied, the loss of gradation is shown at once by the lower elevation of the sand surface after washing.

In the experiments just described, the total depth of sand placed in the filters was 32.5 in.; after removing 3½ in. by scraping, the depth of the sand bed, following a wash at the rate of 26 in. per min., was 30¾ in., and, after a wash at the rate of 10.5 in. per min., this depth was 29¼ in.

The gradation effected by high-rate washes suggests that a reasonable specification for filter sand would define the effective size of the surface layer after washing, and would permit a higher coefficient of uniformity than has been generally required in the past, or one which would result in the bottom layer of sand grading more evenly into the top layer of gravel.

GEORGE A. JOHNSON,* M. AM. SOC. C. E. (by letter).—This paper presents some very instructive data based on experiments which obviously were made with painstaking care. Cincinnati was the first large city to adopt the high-velocity wash idea, although in numerous small plants this method has been practiced for many years. The New York sectional wash filter, which has been used for more than 20 years, embodies the same principles, and the old Hyatt shot-filled strainers were the forerunners of the type of filter bottom described by the authors. It was their purpose, also, to overcome the undesirable feature of gravel and sand mixing and to obtain even distribution of the wash-water; and these were the chief objects sought in the Cincinnati work. Hopper-shaped troughs, running lengthwise of the Mr. Johnson.

* New York City.

Mr. Johnson.

filter tank, the channels being covered with perforated brass strainer plates, is an idea first patented by Hyatt 17 years ago.

The upward velocities of flow of wash-water used by the authors, namely, 15 gal. per sq. ft. per min., are about double those commonly used where compressed air is applied preliminary to the actual washing of the filter. In some places, where relatively high-velocity washing is used—Akron, for example—a rate of 10.6 gal. per sq. ft. per min. appears to be giving satisfaction. At other places the wash-water is applied at a rate represented by about the mean of these two.

The omission of compressed air agitation, as an aid in the washing process, has never been looked on by the writer with unqualified approval. It is his belief that some form of scrubbing of the sand grains usually should precede the application of the wash-water. When treating waters containing large quantities of organic matter, the sand grains become heavily coated therewith, and, if these films are not rubbed off, the sand layer becomes very much lighter, and, even with moderately high wash-water velocities, considerable sand is lost during the washing process. Furthermore, the tendency is greater for mud balls to form, and these, being lighter than the sand, but heavy enough to resist moderately high wash-water velocities, are not removed from the filter, and, growing in size and weight, sink into the interior to clog the filter in spots and thus serve as a direct cause of unequal rates of filtration and wash-water application.

Air agitation, or mechanical stirring, serves to rub the sand grains together far more vigorously than the application of wash-water alone at high velocities. If it were not for the mud-ball feature, common in waters which contain large quantities of vegetable organic matter, the agitation afforded by the high-velocity wash probably would almost always prove adequate; but, in the treatment of some waters of this class, the writer believes that to dispense with the "scrubbing" phase of filter washing is simply paving the way for the complications previously noted.

Mr. Barbour has mentioned this feature, as observed at the Akron plant, and that the sand beds, through the agency of the vegetable matter in the raw water, become more bulky. Consequently, they must become lighter, and the vertical rise of the sand layer during washing must be correspondingly greater. This must mean a waste of sand, unless the critical velocity of the applied wash-water is watched sharply and never allowed to become great enough to permit sand to be carried from the filter. Such restrictions as this may reduce the washing efficiency materially, by allowing balls of this vegetable matter to form and remain in the bed. The writer believes decidedly that, in the treatment of some waters, air agitation, preliminary to the washing process proper, is necessary.

JAMES W. ARMSTRONG,* M. AM. SOC. C. E. (by letter).—At the time Cincinnati was considering the question of the omission of wire screens between the filter gravel and filter sand, and before the experiments mentioned in this paper were undertaken, plans were in preparation for the Montebello filters at Baltimore, Md. Mr. Armstrong.

In order to determine whether or not it would be possible to operate the filters successfully without placing wire screens between the gravel and sand layers, a small wooden filter tank, Fig. 38, was built, early in 1913, somewhat similar to that at Cincinnati. It was 3 ft. by 3½ ft. in plan, and 9 ft. deep. The wash-water trough was 5 ft. 6 in. above the bottom of the tank. Wash-water was admitted to the filters through a wrought-iron pipe manifold with 1½-in. laterals 6 in. from center to center. The laterals had $\frac{7}{32}$-in. holes drilled in them 2½ in. from center to center.

The results of the Baltimore experiments practically coincide with those obtained by Messrs. Ellms and Gettrust, and, in connection with their carefully prepared paper, are of interest only from the fact that, in a number of cases, the wash-water was applied at a much higher rate.

The first few runs were made with both gravel and sand in place. The gravel was 14½ in. deep, and well graded between 3 in. and $\frac{3}{16}$ in. in diameter. There was a depth of 24 in. of sand on top of the gravel. It was found that when the wash-water was passed upward through the gravel and sand at rates giving a vertical rise as high as 2 ft. per min., there was no loss of sand, but when the rate gave a vertical rise of from 2.5 to 2.8 ft. per min., considerable sand was lost.

As the experiments were conducted for the sole purpose of determining the action of the gravel layers under high rates of wash, further experiments with the sand were discontinued, the gravel was re-arranged in several ways, and runs were made with the holes in the manifold system turned toward the bottom of the tank, and others were made with the holes turned upward so that the jets of water impinged directly on the gravel in a direction tending to lift it.

The water for washing was supplied through a 3-in. pipe, directly from the high-service pumps, at a pressure of from 115 to 125 lb. per sq. in. The water was measured with a 3-in. Gem meter, and was applied at rates giving a vertical rise varying from 2 to 6 ft. per min. In no case was there a movement of the top gravel layer sufficient to create any permanent disturbance. After the water had ceased flowing, the gravel returned immediately to its former position, and no evidence existed that there had been any disturbance.

A glass plate in the side of the tank permitted observation of the top surfaces of the gravel and the influence of the water jets on it during the period of washing. Details regarding the depth and size

* Baltimore, Md.

Mr. Armstrong.

of the different gravel layers, the rate of wash and the effect of the water on the gravel surfaces are indicated in Table 8.

EXPERIMENTAL FILTER TANK AT BALTIMORE, 1913.

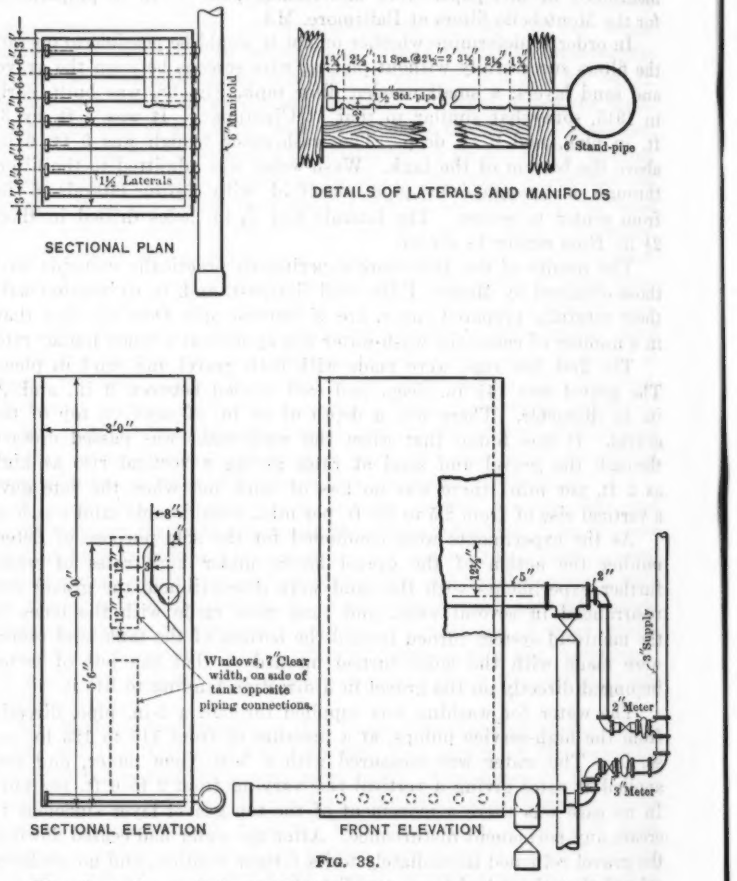


FIG. 38.

The facts learned from these experiments were sufficient to convince those in charge of the design that the wire screens could be safely omitted, and the Montebello filters were accordingly built without them. There was a depth of 14 in. of gravel and 24 in. of sand in each of the filters.

TABLE 8.—ACTION OF GRAVEL AT MONTEBELLO FILTERS, BALTIMORE, MD., WITH VARIOUS RATES OF WASHING. DECEMBER 30TH, 1913. Mr. Armstrong.

Series.	TOTAL THICKNESS OF GRAVEL, IN INCHES.											Laterals opening upward or downward.	Rate of vertical rise, in feet per minute.	Movement of gravel.
	Passing 3 in.	Retained 1 in.	1 "	⅝ "	⅝ "	⅜ "	5/16 "	No. 10.	1 in.	3 "	1 "	⅝ "	7/16 "	¼ "	No. 10.			
				⅝ "	⅜ "	⅜ "	5/16 "	3/16 "	3 "	1 "	⅝ "	7/16 "	¼ "	¼ "				
1	8	3	2	1½	0	....	....	....	....	....	....	Downward	1.51	No movement.
													1.78	No movement.
2	4	1½	1	⅝	1¼	....	....	....	....	....	....	Downward	4.68	Boiling, in jets, rising about ½ in. extreme cases.
													6.28	Boiling localized but violent, 1 in. extreme cases.
													5.20	Boiling gently—isolated grains rise.
													3.73	No apparent motion except at long intervals.
3	4	1½	1	⅝	1¼	....	....	....	....	....	....	Upward ..	3.14	Slight shifting of grains.
													3.75	Slight agitation near glass.
													4.92	Boiling general—½-in. rise in extreme cases.
4	....	....	....	....	....	7¾	3		7/8	2⅞	1¼	Upward ..	1.47	No movement.
													3.51	Slight vibration of finer particles.
													3.96	Particles of mica rise—slight boiling.
													4.84	Boiling over entire area—rising 5/16 in.
5	....	....	....	....	....	7¾	3		7/8	2⅞	1¼	Upward ..	2.47	No movement.
													2.99	No movement.
													3.07 } 3.43 }	Slight sidewise motion.
													3.93	Particles on point of rising.
													4.20	Like gently boiling water.
													5.34	Boils intermittently in spots.
6	....	....	....	....	....	7	2½	1¾	1½	1		Upward ..	4.31	Considerable boiling over entire area.
													4.04	Bubbles in well-defined jets.
													3.94	Bubbling ceases—slight vibration.
													4.23	Bubbles in jets.
													4.26	Bubbling general and violent.
7	....	....	....	....	....	7	2½	1¾	1½	1		Upward ..	4.29	Bubbling begins intermittently.
													4.70	Jetting becomes general.
													5.29	Boiling not severe—5/16-in. rise in extreme case.

Messrs. Ellms and Gettrust.

JOSEPH W. ELLMS,* M. AM. SOC. C. E., and JOHN S. GETTRUST,† ESQ. (by letter).—The writers are gratified that this paper has been so well discussed, and that it has brought forth so much valuable and hitherto unpublished information. Certain features of filter design not directly related to the subject matter of the paper have been brought into the discussion, and have served to heighten its interest. The close relationship between type of strainer system, method of agitation, and character of filter bed is evident from a careful reading of the experimental data submitted.

Mr. Fuller's experience with breaking strainer plates and hook-bolts, coupled with the writers' experience in the breaking of hook-bolts, would appear to indicate that, in the design of this type of strainer system, too little attention has been paid to the stresses created by the upward flow of the wash-water, and especially to those stresses developed at the instant of opening the wash-water valve.

Mr. Manahan seems to be of the opinion that the wearing away of the brass wire-cloth, by the movement of the sand particles through it, had as much to do with its deterioration as did corrosion. It is probably true that the location of the screen subjected it to considerable erosion, and to some strain produced by the pressure of the wash-water on those portions of the cloth where the meshes had become clogged gradually by the lodgment of fine particles of gravel in them; but it is the writers' belief that actual solution of the zinc and copper of the brass by the water passing through the screen had more to do with its destruction than the mechanical agencies. A close examination of the wire-cloth indicates a change in the character of the alloy. It is true that some corrosion of the strainer plates themselves has taken place, but, as they are of much heavier metal, and as they are not subjected to mechanical abrasion in any way, their life has not been affected materially.

The greater latitude permissible in specifications for filter sand which is to be subjected to high-velocity washing seems to be the opinion of both Mr. Stephenson and Mr. Barbour. Mr. Barbour's method of specifying sand for the Akron filter plant seems to the writers a most sensible one, and worthy of general adoption.

The valuable data submitted by Messrs. Willcomb, Barbour, Pirnie, and Armstrong are in accord with the information secured by the writers, and in general their conclusions are the same. Mr. Pirnie's conclusion: "If a filter can be constructed so that the wash-water has a uniform velocity when it passes from the gravel into the sand, there will be no tendency of the gravel and sand to mix", is sound, and sums up concisely the desirable condition to be attained. His further deduction, that all ridges are useless, the writers are inclined to

* Cincinnati, Ohio.
† Kent, Ohio.

believe is correct. All the dispersion effects, which would contribute toward destroying jet action from the strainers, may be obtained by directing the discharge from the strainers toward the bottom or floor of the tank, and by a sufficient depth of properly graded gravel over the strainer system. In the Wheeler bottom, as described by Mr. Barbour, we have as complete a destruction of the jet action of the wash-water, when the jet is directed upward, as has as yet been designed. How the rather broad ridges—necessitated by the construction of the inverted pyramids which hold the balls—will affect the effectiveness of washing in the section immediately above them is worth observing in filter plants using a bottom of this type. Messrs. Elims and Gettrust.

Mr. Armstrong's experiments are of particular interest in showing the stability of the gravel layer under very high rates of washing. These rates are much higher than would ever be required in practice.

Messrs. Willcomb, Pirnie, and Johnson advance some reasons for favoring the use of an air-wash, and seem to think that it will meet certain conditions better than a high-velocity wash. Mr. Pirnie seems to think that the escaping air will break up lumps that form on the surface of the sand, which a high-velocity wash will not do. Mr. Johnson contends that some form of scrubbing action should precede the application of the wash-water, and that the escape of the air will do much toward preventing the formation of mud balls.

The writers' observations on agitation with compressed air are not in accord with these conclusions, and though admitting that the escape of air from a filter bed covered with a layer of water seems to produce an agitation of the sand particles, there is in reality no movement of them, or at the most only a very slight movement of the very finest particles at the surface. Any deposited colloidal coating on the sand, of course, will be broken, and will be projected in part into the overlying water; but the same effect is also produced by the rising currents of wash-water as they leave the channels between the sand grains, break through the colloidal coating on the surface, and enter the main body of the water above the sand. Scrubbing or the rubbing of the particles against each other does not take place when air is forced into the bed in the usual way prior to the application of the wash-water.

The rubbing of the sand particles against each other does take place, however, when the bed is floated by the high-velocity method of washing. In fact, the chief reason for adopting that method in the design of the Cincinnati filtration plant was, that the earlier experiments, made by one of the writers, showed that this method produced this scrubbing effect, and in a measure reproduced the action of mechanical rakes.

The formation of mud balls in filter beds, and the adhesion of the dirty sand to the sides of the tanks were common enough phe-

Messrs. Ellms and Gettrust.

nomena before the high-velocity method of washing was developed, and, consequently, their occurrence in filters cannot be charged to this particular method of washing. Mud balls in the sand, or mud banks on the sides of the tanks, or hard areas in the sand bed, are due to an insufficient quantity of wash-water, or to none at all in the spots affected. This condition may arise from the improper distribution of the wash-water, which must be due primarily to the type of strainer system used, and to the manner in which the rising wash-water is distributed after leaving the strainer openings. If forcing compressed air into the sand bed has any agitating effect at all on the coated sand grains that lay at or near the surface—as some appear to believe, but which the writers have failed to detect—it is undoubtedly true that the application of wash-water at a rate sufficient to float the sand bed effects a very much greater scrubbing action of the sand grains, and all are agreed that this is the desirable effect to be produced by any method of agitation that may be used.

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Paper No. 1370

THE EFFECTS OF STRAINING STRUCTURAL STEEL AND WROUGHT IRON*

By Henry S. Prichard, M. Am. Soc. C. E.

With Discussion by Messrs. J. A. L. Waddell, Henry B. Seaman, C. A. P. Turner, T. D. Lynch, James E. Howard, H. F. Moore, F. N. Speller, J. A. McCulloch, Ernst F. Jonson, T. E. Stanton, Henry M. Howe, A. C. Irwin, David A. Molitor, Clement E. Chase, and Henry S. Prichard.

Synopsis.

Iron, in combination and mixture with small proportions of carbon and other substances, and variously treated in its manufacture, is produced for commercial purposes in many varieties, which differ greatly in strength and ductility but little as regards the rate at which, within the elastic limit, dimensions and bulk are changed by straining at ordinary atmospheric temperatures. One variety, made by welding plastic masses which contain very little carbon and some slag, is termed wrought iron. This is a ductile variety, comparatively low in strength. Most of the other varieties are known as steel. One now little used variety of steel, slag-bearing or weld steel, is made from wrought iron by heating and adding carbon, but without remelting. The other varieties of steel are slagless and, when not simply cast, are made from steel castings (termed ingots) by mechanical and thermal treatment. The different varieties of steel cover a large range of strength, ductility, and hardness, the comparatively soft varieties being more ductile than wrought iron, and the very softest being slightly weaker.

* Presented at the meeting of March 1st, 1916.

The changes in dimensions and shape which take place in pieces of steel or wrought iron when they are strained are brought about either in the first of two certain ways, or in both of these ways in conjunction, depending on the severity of the straining. When strained in the first way, the changes in dimensions, shape, and bulk of a strained piece are effected entirely by changes in dimensions, shapes, and bulks of the particles of the piece, without any sensible slipping of one particle over another. A piece thus strained, on the cessation of the forces which caused the straining, returns to its original size and shape in every part and in every particular; hence the piece is not over-strained and, within the limits of straining within which its each and every part acts in this way, is perfectly elastic. The second way in which changes are effected is by the sliding of particles one over the other under shearing forces, whether the forces are simple shears or the shearing components of tension or compression. Such sliding is caused by straining the metal beyond its capacity to resist shear, and such straining is called over-straining. The changes in dimension and shape resulting therefrom are permanent; but the sliding particles, excepting metal between the sliding surfaces, remain elastic within themselves, and return to their former dimensions and shapes, though in new positions and with changed inclinations, on the cessation of the forces which strained them; hence an over-strained piece is still partly elastic, and tends, to that extent, to recover its original dimensions and shape on the cessation of straining forces.

The elastic limit of steel and wrought iron, though expressed in cases of direct stress with respect to said stress, is determined by the limit to shearing resistance, and is the same in tension and compression, for the reason that the shearing components are the same.

In ordinary testing of structual steel and wrought iron, in tension or compression, the first indication that the elastic limit has been exceeded is usually a breaking of the mill scale, when there is any, followed by a sudden and pronounced yielding. The point at which the pronounced yielding occurs is known as the yield point. When micrometers are used to measure the elongations or compressions, a readily appreciable but lesser yielding can usually, but not always, be observed prior to and within a few thousand pounds per square inch of the yield point. Graphically represented, this preliminary

yielding is, roughly, in the form of a curve tangent to the line representing elastic elongation or compression. The beginning of this preliminary yielding is termed the "proportional elastic limit", or simply the "elastic limit", which is the older, more familiar, and better designation.

When measurements are made with great precision on long gauged lengths, slight imperfections in elasticity, due to initial internal stresses developed during and by the processes of manufacture, can usually be detected at stress intensities far below the point known as the elastic limit in pieces of steel or wrought iron strained for the first time; but, in pieces subjected to only one kind of stress, these imperfections are not important, as the permanent sets in such pieces are very small and the elasticity can be perfected to the elastic limit by straining to that limit. In pieces subjected to alternate direct tension and compression, the elasticity can similarly be perfected; but, if the alternations are rapid and continuous, it cannot be perfected to as high a limit.

When a piece of steel or wrought iron is over-strained, the metal between the surfaces of the sliding particles is viscous, during the over-straining action, in the popular sense of being sticky; that is, it adheres to the adjoining solid particles and has great tensional resistance, and—in the scientific sense of flowing under force but offering resistance as regards the velocity of flow—the flow increases with the intensity of the force and decreases with the degree of viscosity, being very slow when the viscosity is very great. The viscous metal, like the substance pitch, is a liquid, judged by its properties, but, like pitch, is solid in its appearance. The energy which makes and keeps the metal liquid is not in this case applied as heat, but as mechanical force.

Any film of steel or wrought iron made viscous by over-straining at an ordinary atmospheric temperature is at a temperature far below its normal freezing point, and, when released from the shear which makes it viscous, resolidifies with great rapidity. There is considerable evidence to show that after the film has resolidified it is amorphous, like glass, and not crystalline in its structure, and experiments lead to the conclusion that in its supposed amorphous condition it is much stronger than in the original crystalline state in which the solidification had been gradual.

Over-strained steel or wrought-iron pieces suffer temporary loss of strength while the viscous metal is resolidifying; but, as the viscous metal resolidifies with great rapidity on the release of stress, an over-strained piece regains most of its elasticity within a few minutes after the removal of the external forces. If, however, the straining has been much above the yield point, the elastic solid portion of the metal in its recoil will act on the viscous portion and prevent complete solidification for hours or even days and weeks.

When steel or wrought iron is ruinously over-strained, the solid particles will tear apart instead of simply sliding one over the other, and will have to be reheated, or, in severe cases, entirely remelted, before they can again be united. Small tears, not in themselves discernible, will be extended by long-continued or oft-repeated stresses until they cause rupture. This phenomenon is known as fatigue. The tears start where the metal is weakest, presumably at flaws, of which there are probably many of microscopic size in every piece of steel and iron. Experiments and experience indicate that during the life of structures, as determined by other considerations, failure from fatigue does not occur in steel and wrought iron of good quality under repeated stresses less than the elastic limit; but experiments also indicate that such failure is likely to occur under a long-continued succession of rapidly alternating stresses of intensity less than the elastic limit.

For the conditions of ordinary railway bridge practice, the provision for alternate stresses specified by the American Railway Engineering Association seems to be reasonable.

With proper treatment, steel or wrought iron can have its elastic limit raised by over-straining, as in cold-drawn and cold-rolled steel or iron, considerably above the yield point of the ordinary hot-rolled or forged metal; this fact is of great advantage to the manufacturer, and is useful in adjusting and increasing the capacity of some structural parts subjected to bending stresses, but of little value to tension and compression members in service, as the great elongation or compression when a member is strained above the yield point will cause failure of the member, or the entire structure, from distortion, and thus preclude the possibility of subsequent benefit from an enhanced elastic limit.

Fatigue formulas, so-called, have been devised, which are intended to give the greatest strength which can be developed in a piece of steel or wrought iron by straining without resulting in ultimate failure from fatigue. As the greatest strength of steel or wrought iron under repeated stresses is beyond, and in some cases far beyond, the yield point, such a formula, even if perfect, would not be applicable to the design of structural tension and compression members.

General Description of Steel and Wrought Iron.

Steel and wrought iron are essentially iron containing carbon in quantity not exceeding about 0.3% in wrought iron, usually less than 0.9% in steel, much less in structural steel, and not exceeding 2.2% in any steel. Other constituent and foreign substances are always present, some designedly and some unavoidably, some beneficial and some harmful.

Wrought iron differs from slagless steel in the limit to the percentage of carbon, in containing slag, and in being produced by the welding together of small slag-bearing masses, instead of being cast from a melt before rolling or forging. It differs from slag-bearing or weld steel, a now little used kind, in containing less carbon. Steel and wrought iron have a crystalline structure in which the crystalline formation has been more or less interfered with by treatment.

In general, treatment of steel and wrought iron is chemical, metallurgical, thermal, and mechanical. Mechanical treatment is possible because these metals are plastic; that is, they are capable of being more or less moulded by force, even when cold.

Whenever structural steel and wrought iron are strained beyond their elastic limits they are thereby treated to their benefit or injury.

The effect of stress in a piece of steel or wrought iron is to alter its size and shape temporarily, if the stress is within the elastic limit, or permanently, if beyond it, and, if beyond it, to alter the properties of the metal.

Elasticity.

Elasticity, as applied to solids, is the property of returning to original size and shape on removal of stress.

Proportionality of alteration in size and shape to stress is not essential to the conception of perfect elasticity, but experiments indi-

cate that when the elasticity is perfect and the stress is simple tension or compression, such proportionality actually exists. When the stress is compound, the alteration in length in the direction of any one of the stresses is not the same as it would be under a simple stress in that direction, but if all the stresses are increased or decreased in any given proportion, within the elastic limit, the alterations in size and shape will be in the same proportion.

As regards elasticity, steel and wrought iron are isotropic; that is, when their elasticity is perfect, it is the same in all directions around any point.

The ratio of simple stress to alteration in length is expressed by a factor called the modulus of elasticity; this may be defined as one million times the simple stress per square inch required to increase or decrease the length of a bar the one-millionth part of its length.

Many carefully conducted experiments indicate that, at ordinary atmospheric temperatures, the modulus of elasticity of slagless steel, in either tension or compression, does not vary much from 30 000 000 lb. for all grades (this is the technical way of stating that it takes about 30 lb. per sq. in. to stretch or compress such steel the one-millionth part of its length), as shown in Table 1. Wrought iron, doubtless on account of the included slag with which the iron is intimately associated, has a somewhat lower modulus, and the same is probably true of slag-bearing or weld-steel, which is made, what little there is of it, from wrought iron without melting.

There are numerous tests in which the recorded alterations in length seem to indicate erratic variations of considerable magnitude in the modulus of elasticity, but they are probably either in error, or, on account of disturbing elements, are not applicable to the determination of the modulus. Additional and conclusive experiments in this regard are a desideratum.

Simple longitudinal tension, in addition to causing longitudinal elongation, causes lateral contraction; and simple longitudinal compression, in addition to causing longitudinal shortening, causes lateral expansion. The ratio of contraction or expansion, perpendicular to the direction of the stress, to elongation or shortening in the direction of the stress, is termed Poisson's ratio. If the bulk of a body is assumed to increase with pull and decrease with pressure, applied in either one or all directions, Poisson's ratio must be less than $\frac{1}{4}$. For

instance, if equal pressures were applied on the six faces of a cube, and Poisson's ratio were more than $\frac{1}{2}$, then the expansion in the direction of each pair of forces from the combined influence of the other two pairs would be more than $\frac{1}{2}$ plus $\frac{1}{2}$ of the shortening, or greater than the shortening, thereby increasing all the dimensions and the bulk of the cube; or, if Poisson's ratio were just $\frac{1}{2}$, there would be no increase or decrease; but, if less than $\frac{1}{2}$, there would be a decrease in bulk.

TABLE 1.—Comparative Tests in Tension and Compression of $1\frac{1}{4}$-In. Diameter Open-Hearth Steel Bars of Various Grades, Turned to Specimens from that Size.

Compression Specimens 1.0092 in. Diameter, 12 in. Long.
Tension Specimens, 1.0092 in. and sundry Diameters; Gauged Length, 30 in.

Tests made at the Watertown Arsenal, and given in the Reports for the Year 1886, pp. 1635-1653; and for 1887, pp. 802-822.

Marks.	Composition, Percentage of:			Modulus of Elasticity.		Elastic Limit.		Yield Point.*	
	C.	Mn.	Si.	Compression, in pounds per square inch.	Tension, in pounds per square inch.	Compression, in pounds per square inch.	Tension, in pounds per square inch.	Compression, in pounds per square inch.	Tension, in pounds per square inch.
833	0.09	0.11	....	30 120 000	30 151 000	30 500	30 000	32 125	30 500
123	0.20	0.45	....	30 303 000	30 151 000	37 000	39 500	39 190	39 500
782	0.31	0.57	....	30 612 000	30 000 000	44 500	46 500	45 500	46 500
795	0.37	0.70	....	31 250 000	30 151 000	47 000	50 000	50 875	50 000
805	0.51	0.58	0.02	30 075 000	30 000 000	57 000	58 000	58 000	58 000
797	0.57	0.93	0.07	30 201 000	30 104 000	55 500	55 000	Not defined.	56 000
828	0.71	0.58	0.08	31 034 000	30 088 000	55 500	57 000	"	58 000
750	0.81	0.56	0.17	30 000 000	29 923 000	74 500	70 000	"	Not defined.
756	0.89	0.57	0.19	30 612 000	29 864 000	76 500	75 000	"	78 000
334	0.97	0.80	0.28	30 822 000	29 817 000	83 000	79 000	"	80 000

* The yield points are not specifically noted in the records, but in most cases are indicated: in tension, by increase in elongation under reduced loads and by great differences in elongations recorded for slightly differing loads; and, in compression, by buckling to failure after a slight increase in rate of shortening.

Poisson arrived at the existence of this contraction or expansion, at right angles to pull or pressure, from theoretical considerations; and his theory led him to assign to his ratio a value of $\frac{1}{4}$, but his theory has been questioned. Experiments indicate that the value for steel is between $\frac{1}{4}$ and $\frac{1}{3}$.

The behavior of an isotropic elastic solid body, strained within the limits of its elasticity, is entirely determined by its modulus of elasticity and Poisson's ratio. Writers on theoretical mechanics have found it convenient to introduce other constants, but they are not needed for the purposes of this paper.

Elasticity is one of the most useful properties of matter. Without it, structures and machines would be impossible. It is the foundation on which many useful engineering theories are based, and it affords the mathematically inclined endless possibilities for intricate analysis, but to the practical engineer the most important question regarding it is its limit. The limits to the elasticity of steel and of wrought iron are closely related to the resistance of these metals to shear.

Shear.

The internal forces acting on any plane within a body can be resolved into a tension or compression at right angles to the plane—that is, normal to it—and a shear tangential to it.

When the force is a simple pull or pressure on a bar, as in Fig. 1 (drawn for pressure), the tension or compression has a maximum intensity, and the shear is zero, on a plane normal to the direction of the force; and the shear has a maximum intensity on a plane making an angle of 45° with the normal plane.

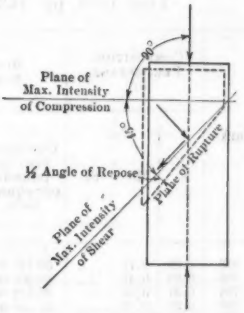


Fig. 1.

Experiments show that materials which are brittle or comminuible, "when subjected to a compressive load, fail by shearing on certain definite angles. The resistance to movement along these angles is made up of two parts; first the strength of the material to resist shearing; and second, the frictional resistance to motion along this plane." Analysis leads to the conclusion that rupture should take place on a plane making an angle with a normal plane of 45° plus one-half the angle of repose of the material. The analysis on which this conclusion is based* is supported by the results of numerous experiments and photographs of the specimens after rupture.

* J. B. Johnson's "Materials of Construction", pp. 24–28.

Under pressure, plastic materials, like structural steel and wrought iron, do not fail by shearing on any one plane, like the specimen in Fig. 1; but, after the elastic limit is passed, they simply spread, as in Fig. 2. Under pull, after the elastic limit is passed, they draw out like taffy, as in Fig. 3, to a greater or less extent, before rupture.

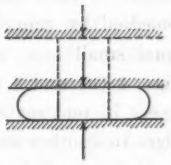


FIG. 2.

The fact that the deformation of materials depicted in Figs. 2 and 3 was caused by failure to resist shear is not evident from simple observation.

The usual conception of movement under shearing stress is that of sliding, but there is no visible evidence of the sliding of one particle on another before the elastic limit is reached. Change in shape within the elastic limit is not due to any sliding over one another of the individual grains or crystals; instead of this, each grain, no matter how minute, is itself deformed. What takes place among the molecules of the material under shear is a matter of speculation.

FIG. 3.

When the elastic limit is exceeded, the ensuing deformation is caused largely by actual sliding of the particles.

"These slips are seen under the microscope as sharply defined lines which appear on the polished surface of each grain as soon as the yield-point in any process of straining has been reached."*

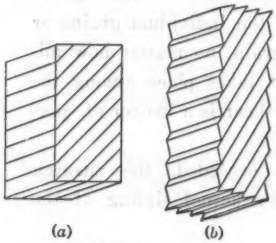
Of course, it is impossible to know or illustrate just how the particles move and slide in deformation beyond the elastic limit, but equivalent movements and slips can be conceived to illustrate how the geometry of such deformation can be accounted for on the hypothesis of sliding and rotating particles, as follows:

A square bar, made up of layers inclined at an angle of 45°, as in Fig. 4 (*a*), could be elongated longitudinally and contracted laterally by rotating the layers and sliding them over one another, as in Fig. 4 (*b*). Two of the sides of such a bar would not be plane, but, if the layers were very thin, the surfaces would appear to be plane with fine markings.

* James Alfred Ewing, in Encyclopædia Britannica, 11th Ed., Vol. XXV, p. 1018.

Such a bar would be contracted in only one lateral direction, whereas bars under simple pull are contracted in two lateral directions perpendicular to each other. This defect in the supposititious bar can be remedied by conceiving the bar as divided into a large number of equal small bars, each, in turn, divided into inclined layers, as in Figs. 4 (*a*) and 4 (*b*), but grouped so that the inclined edges of the layers in one set of small bars would be adjacent to the horizontal edges in another set, as in Fig. 5.

If the small bars into which the large bar is conceived as divided are very small, and the surface of the large bar is conceived to be polished, the edges of the layers would appear as straight lines. The angles which these lines would make with a plane normal to the direction of the pull would have a tangent equal to $\frac{1}{2}$ tan. 45°, or 0.5,



(*a*) (*b*)

FIG. 4.

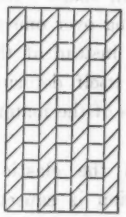


FIG. 5.

which corresponds to an angle of 26° 34′. Such lines are known as Luder's lines. Their occurrence in pulling tests, as indicated in specimens of full thickness by the breaking of the mill scale, is a very common phenomenon. It is evident from simple inspection that the angle these lines make with a plane normal to the direction of the pull is usually much less than 45° and often not far from the angle of 26° 34′ as just deduced. Before this deduction was made, Professor Ewing* stated that this angle had been found to be about 25 degrees. Two sets of Luder's lines, sloping in opposite directions, appear in tests, which shows that the actual condition is more intricate than the equivalent condition assumed. The condition assumed, however, makes it evident that the lines do not directly indicate the slopes of the planes on which slipping occurs, but instead, the mean inclinations of the edges of the planes.

* Encyclopædia Britannica, 11th Ed., Vol. XXV, p. 1016.

Lateral expansion under longitudinal pressure can be accounted for in a manner similar to that for lateral contraction under longitudinal pull.

The foregoing explanation of deformation under pull raises the question: What holds the grains together in tension after they sever their original connections and rotate and slide? The answer involves a consideration of the nature of the change which occurs in the condition of steel and wrought iron when the limits to elasticity are exceeded.

The Elastic Limit.

Material which can be constrained by mechanical force into new forms without breach of continuity is said to be plastic; hence, wrought iron, soft steel, and, to some extent, even hard steel, are plastic. They are seldom characterized as plastic, but wrought iron and soft steel are described as malleable and ductile, which terms imply a great range of plasticity. Some of their characteristics while flowing are those of a liquid.

Liquids have no static resistance to shear, but have cohesion* among their molecules, which is nil at their boiling point and increases toward their freezing point.† Cohesion without static resistance to shear does not prevent but does impede flowing, and this property is termed viscosity. Liquids, however, are not usually termed viscous unless they possess a high degree of viscosity. Viscous liquids not only have cohesion, but, to a greater or less extent, have the property of adhering to solids. This useful property, this glue-like stickiness, though not essential to the scientific conception of viscosity or included in the scientific definition of the term, is firmly ingrained—in fact predominates—in the popular idea of the meaning of the word. Above certain temperatures, and at ordinary temperatures under certain mechanically produced conditions, viscous portions of structural steel and wrought iron adhere strongly to the solid portions of the metal.

According to some physicists, the plastic solid is distinguished from the viscous liquid in that the former requires a certain magni-

* Newton devoted the 31st query in the last edition of his "Opticks" to molecular forces, and instanced several examples of cohesion of liquids, such as suspension of mercury in a barometer tube at more than double the height at which it usually stands.

† The surface tension diminishes as temperature rises, and when the temperature reaches that of the critical point at which the distinction between a liquid and its vapor ceases, it has been observed by Andrews that the capillary action also vanishes. James Clerk Maxwell, and Lord Rayleigh, in Encyclopædia Britannica, 11th Ed., Vol. V, p. 261.

tude of stress to be exceeded to make it flow, whereas the viscous liquid will yield to the slightest stress, but requires a certain length of time for the effect to be appreciable. This distinction is generally accepted, but is superficial.

The change from a solid to a liquid state is a radical one. Considerable energy, usually in the form of heat, is required to melt the solid, which energy is not expended in raising temperature, but in effecting molecular changes by reason of which the potential resistance to flowing is overcome and the molecules are less restricted and have greater freedom of action. It is this difference in molecular arrangement, no matter how produced, which is the real distinction between a liquid and a solid.

A certain portion of a plastic metal, when caused to flow by "a certain magnitude of stress", is a highly viscous liquid during the continuance of the flow, and even for a time after the removal of the stress; and it will be so termed in dealing with this subject.

Mr. H. E. Tresca, by applying great pressure, caused copper, silver, iron, and other metals to flow, like viscous liquids, through a small circular aperture at the bottom of a cylinder, and without breach of continuity. Parallel experiments, with layers of dough or sand plus some connecting material, proved that the particles moved along the same tracks as would be followed by a flowing cylinder of liquid.*

When structural steel and wrought iron become partly viscous and begin to flow, the elastic limit, of course, has been reached; and the converse is true: when the elastic limit is reached, structural steel and wrought iron become partly viscous and begin to flow. It is because the viscous portion of the metal acts somewhat like glue does with wood, or china cement with china, that the sliding grains or crystals adhere in their new positions, and finally, after the viscous portion has solidified, form a coherent mass.

When structural steel and wrought iron are strained for the first time after leaving the rolls or forge they are apt to have slight imperfections in elasticity, indicated by slight permanent sets, under loads far below those at which the general yield begins.

Structural parts are strained so frequently that even very slight permanent sets, if repeated and accumulative, would become ruinous

* Encyclopædia Britannica, 11th Ed., Vol. XVIII, p. 199.

in a comparatively short time. For reasons discussed in the next section—The Beneficial Effects of Strain—such increases in permanent sets, under loads below that at which the general yield begins, do not occur, and slight imperfections under such loads are of little practical importance.

Structural steel and wrought iron are not minutely homogeneous with respect to resistance to shear; that is, although small specimens of metal of good quality are nearly alike, microscopic specimens differ; but, for a considerable proportion of the metal within a specimen, the resistance is so nearly uniform that, after a certain point in stress is reached, a general yield develops under a slight increase in stress intensity.

In simple tension or compression, the yield usually culminates in a flow under a constant or decreasing force at a point called the yield point. At this point, in slowly conducted tests, the intensity of the stress frequently decreases several thousand pounds, in extreme cases as much as 10 000 lb. per sq. in.; after which it takes an increasing force to cause the metal to flow with any degree of rapidity. In some tests, the yield point is indicated by a relatively great increase in elongation or shortening under a slight increase in stress.

In ordinary specimen tests, the yield point is usually discernible by the drop of the beam, unless the steel is hard and the speed rapid. The yield point is often called the elastic limit, for the reason that in ordinary testing it is the first indication that the elastic limit has been reached. By some authorities it is called the apparent elastic limit; but yield point is the best name.

The beginning of the general yield sometimes coincides with the yield point; that is, in such cases, the general yield develops instantaneously to a flow without increase in load. More often the beginning of the general yield occurs under a stress from a few hundred to a few thousand pounds per square inch below the yield point, and is indicated by increase in rate of elongation or shortening, which increase is discernible in ordinary specimens by measuring carefully the elongation or shortening with sensitive micrometers.

By some authorities, the beginning of the general yield is called the proportional elastic limit, or the limit to proportionality, for the reason that, below this point, the elongation or shortening indicated in simple tension and compression tests is nearly proportional to

stress, even on the first application of the load. In the detailed reports of the many scientifically conducted tests at the Watertown Arsenal, this point is called the elastic limit; which is the better name for it, for three reasons: first, the elasticity of the metal up to this point can be perfected by straining, as explained in the next section—The Beneficial Effects of Strain; second, the permanent sets below this point do not increase under repeated or continued stresses, and are too slight to be of any practical importance; third, proportionality of elongation or compression to stress is not the essential but simply an observed characteristic of elasticity.

When the gauged lengths are long and the measurements are made with great precision, the true elastic limit, as some authorities term it, is usually very low the first time the metal is strained; in many cases it has been observed to be as low as 5 000 lb. per sq. in. This point, for reasons explained in the next section, is of no practical importance. It is doubtful, in fact, whether, when structural steel and wrought iron are strained for the first time, there is any point below which elasticity and proportionality are absolutely perfect.

The elastic limit, defined as the beginning of the general yield, is the most important measure of the most important property of structural steel and wrought iron, because it is the criterion of permanent strength without, or with only slight, permanent deformation, and because it is the limit to the applicability of those methods of determining stresses and deformations which are based on the proportionality of stresses to deformation.

Experiments have shown that the yield point and elastic limit of structural steel and wrought iron are practically the same in compression as in tension. As this fact is crucial in explaining what takes place at the elastic limit, the results of pertinent experiments are given in Tables 1 and 2. The tests in Table 1 are part of a series of scientific experiments made by the Government's engineers at the Watertown Arsenal, and are notable for their precision and the wide range of grades covered—from carbon 0.09 to 0.97 per cent. The tests by the late Charles A. Marshall, M. Am. Soc. C. E., in Table 2, are part of what was probably the most exhaustive comparison of steel and wrought iron in compression with steel and wrought iron in tension. For both of these metals all his results are substantially in accord with those in Table 2.

TABLE 2.—Compression and Tension Tests of Steel Bars, all from the Same Blow of Bessemer Steel "as from Rolls".

Tests made by the late Charles A. Marshall, M. Am. Soc. C. E. *Transactions*, Am. Soc. C. E., Vol. XVII, p. 68.

Pieces for Tension Tests cut from same bars used in Compression Tests. All tests were of full-sized bars, except specimens tension tests of 3-in. bars more than 1 in. thick. Bars tested in compression had flat ends.

$$\frac{l}{r} = \frac{\text{Length}}{\text{Radius of Gyration}}.$$

	Compression.								Tension. Elastic Limit "By Beam"; Gauged Lengths, 8 to 25 Inches.	
	Elastic Limit.	Elastic Limit, "By Beam"†—Which in These Tests was also Failure by Yielding and Bending.								
Length....	2 Diameters.	30 and near 30 Diameters.		24 Diameters.		18 and near 18 Diameters.		12 and near 12 Diameters.		
Size of bar, in inches.	Pounds per square inch.	$\frac{l}{r}$	Pounds per square inch.	$\frac{l}{r}$	Pounds per square inch.	$\frac{l}{r}$	Pounds per square inch.	$\frac{l}{r}$	Pounds per square inch.	Number of tests averaged.	Pounds per square inch.
¾ round.	47 300	....	.....	....	.....	....	.....	50	44 980	4	46 090
1 "	46 020	....	.....	....	.....	....	.....	48	44 000	4	44 202
1¼ "	43 460	....	.....	....	.....	....	.....	48	40 850	3	40 747
1½ "	41 290	....	.....	....	.....	....	.....	48	41 880	3	40 275
1¾ "	42 075	....	.....	....	.....	....	.....	48	39 950	3	40 017
2 "	38 830	....	.....	....	.....	....	.....	48	40 850	3	38 207
2¼ "	38 125	....	.....	....	.....	72	36 580	48	36 790	1	37 000
2½ "	36 840	....	.....	....	.....	70	34 450	48	35 650	1	36 100
¾ square	43 845	....	.....	....	.....	....	.....	45	44 960	3	44 273
1 "	44 025	....	.....	....	.....	....	.....	42	43 080	3	43 560
1¼ "	42 300	....	.....	....	.....	....	.....	42	40 060	3	41 060
1½ "	42 740	....	.....	....	.....	....	.....	42	38 420	3	39 317
1¾ "	40 630	....	.....	....	.....	....	.....	42	39 450	3	38 193
2¼ "	39 940	....	.....	....	.....	63	37 330	42	39 270	1	38 310
3 by ⅜....	.....	....	.....	83	46 150	63	46 420	42	47 650	3	47 363
3 " ½....	.....	104	42 170	83	43 490	....	.....	42	41 650	3	44 417
3 " ¾....	.....	102	40 700	....	.....	....	.....	42	43 580	3	41 447
3 " 1	.....	....	.....	83	36 920	....	.....	42	41 200	3	39 397
3 " 1¼....	.....	....	.....	....	.....	63	35 430	39	38 350	4	38 482
3 " 1½....	.....	....	.....	....	.....	....	.....	42	36 920	4	37 820
3 " 1¾....	.....	....	.....	....	.....	....	.....	42	35 760	4	35 917
3 " 2	.....	....	.....	....	.....	....	.....	42	37 670	4	39 302
*4 " ½....	.....	104	51 210	83	55 320	63	55 110	42	55 420	2	53 800
4 " 1	.....	104	39 270	....	.....	....	.....	42	42 630	2	41 415
4 " 1¼....	.....	....	.....	83	38 200	....	.....	42	39 300	1	36 680
4 " 1½....	.....	104	39 100	....	.....	63	39 850	42	39 500	1	37 580
							Average.....		41 147	.....	40 867

Notes.—Results for specimens 2 diameters long are the average of two tests each, except for ¾ in. round and 1 in. square. All other results in compression are those of single tests. "The short compression specimens were removed and measured with a measuring machine." "The tendency is with very short specimens, either in tension or compression, to get too high values for elastic limit."

* "Bar finished at very low heat."

† Elastic limit "by beam" is now frequently termed yield point, which is better practice, as the point properly termed elastic limit and sometimes termed proportional elastic limit is usually somewhat lower.

This equality of the yield point in tension and compression shows that resistance to flowing is just as great when there is a pull normal to the direction of the shear as when there is a pressure. It seems fair, therefore, to infer that flowing occurs just as soon as, and not before, the resistance to shearing is overcome, uninfluenced by the kind (tension or compression) or intensity of the normal component, and irrespective of whether the shear was produced by simple or compound stresses. What seems to be a yielding to simple tension or compression is really a flowing under shear.

The question whether maximum stress, maximum strain (linear), or maximum shear is the critical consideration has been much debated during the last fifteen years. That shear is the critical consideration is supported by the experiments and analysis of J. J. Guest,* Professor E. L. Hancock,† and others. For plastic materials, the conclusion seems inevitable that shear is the critical consideration as regards the elastic limit. Frictional resistance, which in cases of comminuible materials supplements resistance to shear (see page 1436), cannot be much of an element in the flowing of plastic metals, for the reason that the flowing metal is either all viscous liquid or solid particles mixed with and well lubricated by viscous liquid. There is no angle of repose of a liquid; otherwise it could be permanently piled, like sand or coal.

In cases of simple tension and compression, it is the custom to state the elastic limit in terms of the intensity of the tension or compression. This custom is simple, and it is not advisable to attempt to change it, even though it is not tension or compression but shear that is critical. It is well, however, to be able to state for plastic metals the elastic limit in terms of shear intensity, so that it can be generally applied to compound as well as simple stresses. From the fact that the maximum shear occurs in a plane making an angle of 45° with the direction of a simple tension or compression, it can be readily shown that the quantity which expresses the elastic limit in terms of shear intensity is one-half the quantity which, in cases of simple tension or compression, expresses the intensity of the tension or compression at the moment the elastic limit is reached. For instance, if the elastic limit for a certain quality of steel has been definitely found by

* *Phil. Mag.*, July, 1900.
† *Phil. Mag.*, 1906, Vol. XI, p. 276, and Vol. XII, p. 418.

tension tests, and appears to be 36 000 lb. per sq. in. in tension, it really is 18 000 lb. per sq. in. in shear; and, under whatever condition of loading, simple or complex, when the shear reaches 18 000 lb. per sq. in., and not before, the elastic limit in this precise quality of steel will be reached.*

If steel or wrought iron should be pulled equally in all directions, there would be no shear, so that failure, if it occurred, would take place without any preliminary elastic limit; but there is no known way of making such a test.

Hydrostatic pressure, which, of course, is equal in all directions, has been applied to mild steel, wrought iron, cast iron, drawn brass, and drawn copper, in experiments at the Watertown Arsenal.† The pressure was estimated as 90 000 lb. per sq. in., except for one steel specimen for which it was 117 000 lb. per sq. in. The permanent effect of compression on the dimensions of the specimens was not appreciable with the steel, brass, and copper. The wrought iron and cast specimens after testing were smaller, but, as the reduction in dimensions did not take place uniformly, the effect was attributed to the closing of interstices formed by blow-holes, or the presence of cinder. A comparison of the results of tests shows no appreciable change in the strength, elasticity, and ductility of the compressed over the uncompressed metal.

The Beneficial Effects of Strain.

Between the temperatures at which iron and ordinary steel (containing less than 0.9% of carbon) are entirely liquid and the temperatures at which they are entirely solid there is a great range, for the major part of which the metal is technically a solid, because it requires stress exceeding a certain magnitude to produce continuous alteration of form. It does not follow, however, that the metal is all solid. It will answer the description of "plastic" if it consists of an intimate mixture of liquid and solid particles in which the solid particles are sufficiently connected to give the mass strength to resist a limited amount of shear. There are many plastic solids, such as putty, plaster, mud, and modeling clay, which are solid, according to

* This fact should be applied with caution. Quality varies, even within the same piece, and actual intensity is partly dependent on unknown initial internal stress; hence, the nominal stress intensity at which the elastic limit occurs is subject to variation in metal of nominally uniform quality.

† Report for 1886, pp. 1663, 1687.

the usual distinction between a viscous liquid and a plastic solid, but which, nevertheless, are part liquid and part solid. From the fact that changes take place in plastic iron and steel, within a certain range of temperature, which parallel those which take place in the viscous liquid state, it is highly probable that within this range the metal consists of an intimate mixture of solid particles, sufficiently connected to make the mass as a whole solid, and a highly viscous liquid which, like pitch, looks solid and adheres to the strictly solid portion. Be this as it may, a portion of the metal is a viscous liquid when it flows under pressure or pull.

According to metallurgists, as iron and steel with less than 0.9% of carbon slowly cool, the iron solidifies in first one and then another potentially hard, strong form containing carbon in solid solution, like sugar in gelatin, until a stage in the process of solidifying is reached, between 700 and 900° cent. (about 1 290 and 1 470° Fahr.), during which the iron expels the carbon and assumes a softer, weaker, pure form; the carbon combines with iron, forming cementite, Fe_3C, harder than glass and nearly as brittle, but probably very strong; the two substances minutely interstratify, in the proportion of six parts of iron to one part of cementite, and thus form a substance termed pearlite; and the pure iron and pearlite intimately mix. The changes at the end of this stage evolve so much heat that the mass is heated, and brightens in a striking manner. This phenomenon is called the "recalescence".

Slow cooling, through the stage at the end of which recalescence occurs, gives full opportunity for changes in the form of the iron and in the mixtures which result from the presence of carbon, and it facilitates the growth of crystals and of a coarse crystalline structure. These developments can be arrested or modified by hastening the complete solidification by sudden cooling, thereby producing in most cases a stronger steel than is obtained when the metal is allowed to cool slowly. The addition of certain elements, as in some of the alloy steels, will similarly arrest or modify the transformations.

It is not known to what extent steel or wrought iron made viscous by strain is similar in condition to the same metal at some high temperature, but it possesses one of the important characteristics of such hot metal—its sudden solidification makes for hardness and strength.

Structural steel and wrought iron during their manufacture are slowly cooled, undergo one or more reheatings, and are subject to mechanical force while cooling. The metal is an intimate mixture of particles which differ in their capacities to resist shear, though in most cases a considerable proportion forms a matrix which is nearly homogeneous in this respect and has disseminated throughout its mass the more or less connected stronger grains.

All structural steel and wrought iron, before the metal is strained in service or tests, contain in their free state some balanced initial internal shears produced by unequal cooling and the processes of manufacture. Therefore, when strained by an external load, the real intensity of the shear at any point is that of the apparent shear plus the initial internal shear. When this combined shear exceeds the shearing resistance of the metal, the latter becomes viscous and flows, thus relieving the initial internal shear, but causing a permanent set.

When, in such cases, the load is removed, the viscous metal resolidifies at a temperature far below its natural freezing point, and acquires thereby a very much greater shearing resistance, so that if, after a rest, a load in the same direction is again applied, the elasticity will be perfect up to the limit of the former loading. If the loading is carried beyond that limit, it will be some other portion of the metal not previously over-strained which will be the first to yield under the new loading.

In this way steel and wrought iron structural parts not subject to alternate stresses can have their elasticity perfected to the beginning of the general yield—the point known as the elastic limit. This conclusion agrees with Bauschinger's experiments, and harmonizes with the general trend of the experiments at the Watertown Arsenal; but the most satisfactory evidence of its substantial accuracy is the behavior of structures in service which, under frequently repeated and long continued heavy loads, give no evidence of any growth in permanent set so long as the strains are below the point at which the general yield would begin. It is possible that elasticity is similarly perfected to the same point in parts subjected to alternate stresses, but, for reasons explained in the next section—Fatigue—it is advisable to use, for such parts, a somewhat lower point as the criterion of permanent strength.

The elasticity can be perfected by over-straining, and the elastic limit raised thereby, even when the over-straining is carried beyond the original yield point; but this fact is of little practical importance to those structural members which are subjected primarily to direct tension or compression. The elongations and shortenings in such members are so great, after the yield point is reached, as to produce ruinous deformation in the structures of which such members are a part. In the case of compression members, unless they are very short, strain to the yield point means failure, as shown in Table 2.

The raising of the elastic limit by over-straining is of great use in manufacturing steel and wrought iron. It is this which gives to drawn wire, drawn steel, and cold-rolled steel their enhanced strength; and it is partly by over-straining that hot-forging and rolling raise the elastic limit of the metal; though part of the virtue of these operations on hot metal is due to refinement of crystalline structure and to the closing of cavities by bringing their sides to a welding contact. In all these manufacturing processes, to achieve good results, it is essential that the over-straining should not be too great, as explained in the next section—Fatigue.

When strains in steel and wrought iron are produced by transverse loading, there are special features by reason of which the effect of such strains requires separate consideration. These features are as follows:

First.—The distribution of stress, in cases of transverse loading, is too complicated a problem to admit of precise determination. The ordinary theory of flexure, even within the limits of perfect elasticity, is only an approximation.* It neglects the influence of distortion from shear on distribution of stress, and, in consequence, the stress in the extreme fiber is greater, so long as the elasticity is perfect, than the ordinary theory indicates. This is especially the case with short beams, having thin webs, under heavy concentrated loads.

Second.—The beginning of the general yield, as distinguished from imperfections in elasticity due to initial internal stresses, is not well marked.

* Professor A. E. H. Love on Elasticity, in Encyclopædia Britannica, 11th Ed., Vol. IX, pp. 141–160; also, Prichard on "Faults in the Theory of Flexure, and an Epitome of Certain I-Beam Tests made at Ambridge, Pa.", *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 895.

Third.—In consequence of the different distribution of stress, from that which obtains in cases of direct tension or compression, the progress and consequences of the yield are correspondingly different.

Consider the case of a horizontal round or square bar loaded vertically; when the load is such that stresses in the extreme fibers just equal the elastic limit, the intensity of the stresses will decrease nearly uniformly from the top and bottom toward a neutral axis at the center. When the load is increased, the elastic limit will be exceeded in some part of the extreme fibers, and they will become viscous and flow, thus relieving themselves of the stress in excess of such limit. The fibers immediately below the top and bottom ones will then be strained to their elastic limit, and the process will continue, when the load is sufficient, until, for quite a distance from the top and bottom of the bar, the metal at the critical portion of the span will be strained to its elastic limit and develop increased resistance accordingly. Moreover, the viscous metal will resolidify in stronger form. If the load is gradually taken off, the fibers toward the top and bottom will be entirely relieved of their stress before those nearer the center, after which tension will be developed in the top and compression in the bottom, forming a couple balanced by another and opposite couple nearer the axis; moreover, the bar will have a permanent deflection. If the load is again applied, the effect of taking off the load will be reversed, without any additional over-straining, unless the former load is exceeded. In this way, at the expense of a little permanent deflection, the strength of the bar to resist transverse loading in one direction can be greatly enhanced; but, if the direction of the load is reversed, the balanced internal stresses will tend to lower the elastic limit of the bar.

The principles explained in the foregoing paragraph apply to all cases of transverse loading, but, for beams, rolled or built, of I-section, the possibilities of enhancement of strength are not nearly as great as for those of round or square section, and may be offset by spreading the metal too thin.

Fatigue.

Fatigue, as applied to metals, is weakness produced by strain. It may be temporary or it may be permanent, and it may gradually increase under oft-repeated or long-continued stresses until it produces failure.

Structural steel and wrought iron made viscous by strain are extremely viscous; that is, their flow under stress is exceedingly slow. For instance, a wrought-iron bar, 1 in. square, tested as a horizontal beam, which had been made partly viscous by a gradually increasing center load during 8 days, was then loaded at the center with 2 589 lb. Under this load its deflection gradually increased during 61.5 hours from 4.2749 to 4.6701 in., at which point the weights reached the support and the test was discontinued.*

When a bar of structural steel or wrought iron is strained beyond its elastic limit, it becomes an intimate mixture of viscous liquid and elastic solid portions. After the load which produced the strain is removed, the solid elastic portion endeavors to resume its former size and shape, and would do so almost instantly were it not for the opposition of the viscous liquid which adheres to it and is compelled thereby to flow, but which refuses to flow rapidly and, in consequence, is for a while subjected to a push or pull from the elastic solid portion, the return of which to its former size and shape is thus delayed.

The viscous portion of over-strained steel and iron may solidify even while the piece of which it is a portion is subjected to stress. For instance, the 1-in. square bar just referred to was loaded in the center, at one stage of the experiment, with 1 600 lb., and while thus loaded its deflection gradually increased during 16 min. from 0.489 to 0.6598 in., but then the viscous metal appears to have solidified, as there was no further deflection during the remainder of the 5 hours and 44 min. that the load remained unchanged.

Just as there is, for each elementary portion of structural steel and wrought iron at atmospheric temperature, a certain intensity of shear beyond which solid metal becomes viscous, so there is a certain intensity of shear beyond which viscous metal subjected thereto remains viscous and flows; but when the flowing releases the viscous metal from shear in excess of the critical intensity, it will resolidify. In cases where iron and steel have been strained to a point below that at which the general yield commences, but at which, by reason of initial internal stresses, a small portion of the metal has become viscous, the recoil of the large elastic solid portion on release of stress will act on the

* Report of U. S. Board on Testing Iron and Steel, Vol. 1, pp. 464, 469, and 471 (1881).

viscous portion, but with such low intensity and through such a short distance that it will not prevent rapid solidification.

On the other hand, in cases where the yield point has been somewhat exceeded, the stress intensity in the elastic solid portion of the metal will be very high, and, on the release of the stress, this solid portion, in its endeavor to regain its former size and shape, will push or pull some part of the viscous portion, causing it to remain viscous and slowly flow while the solid portion gradually recoils. This process will continue until the intensity of the shear resulting from the push or pull is reduced to a point below that required to prevent the solidification of the viscous portion.

Much of the recovery after over-strain takes place in a few minutes, and nearly all in a few hours; but, toward the very end, as the force from the recoil of the elastic solid portion of the metal approaches the intensity permitting complete solidification, the process becomes exceedingly slow, and may continue for many days. At this stage, it appears, from experiments by Mr. J. Muir, that the shear exceeds the resistance of the viscous portion of the metal to flowing by such a narrow margin that the changes incident to a slight increase in temperature will turn the scale. He found that pieces of medium steel which normally required a long rest to recover their elasticity at atmospheric temperature, after being over-strained in tension, recovered it in a few minutes when exposed to the temperature of boiling water.*

Straining metals beyond their elastic limit and thus causing them to flow, as explained in the section on Shear, implies a rotation of one group of solid elements through a definite angle with respect to another group of such elements. This tends to create minute interstices between solid elements. These interstices have to be filled with viscous metal to act as cement, or there will be cavities and cracks. If iron or steel is not very ductile, so that only a small portion becomes viscous under high stress intensity, or if it is ductile but so overstrained that the viscous portion will not fill the interstices, the overstraining will develop permanent cavities and cracks which nothing but heating will remedy. In fact, iron and steel can be strained, even when at welding heat, to an extent which develops flaws which nothing short of remelting can remedy.

* Muir "On Recovery of Iron from Overstrain", *Phil. Trans.* A, Vol. 193, (1910).

With respect to strain, hot steel (or hot wrought iron) differs from cold steel (or cold wrought iron) in having a lower elastic limit, in offering less resistance to deformation under stresses in excess of the elastic limit, and in affording opportunity for heat treatment and welding while being over-strained. It is possible that some of the alloy steels owe their good qualities in part to an influence in the hot metal whereby it permits and receives greater over-straining under good welding conditions than in ordinary steel.

Blow-holes, foreign substances (such as sulphide of iron and graphite), and cavities and cracks, caused by over-straining while cold or hot, break the continuity of the metal and thereby weaken it. Probably no steel or iron is free from such flaws, though in metal of good quality it requires a microscopical examination to discover them. These breaks in the continuity of the metal cause the stress to be concentrated at their edges, and it is in this way, much more than by reducing the effective cross-sectional area, that these flaws weaken the metal in which they occur.

It is common experience that the easiest way to break a small piece of metal is to bend it alternately backward and forward. Similarly, by straining bars alternately in tension and compression, as in a rotating shaft, a small crack can be extended by stresses locally until it becomes a plane of rupture without much preliminary evidence of a general yield.

"In experiments by Ewing and J. C. W. Humfrey ('The Fracture of Metals under repeated Alternations of Stress,' *Phil. Trans.*, 1903) the microscope was employed to examine the process by which metals break through 'fatigue' when subjected to repeated reversals of stress. The test-pieces were short rods overhanging from a revolving mandrel and loaded at the end so as to produce a bending moment. A part near the support, where the stresses due to bending were greatest, was polished beforehand for observation in the microscope. After a certain number of reversals the surface was examined, and the examination was repeated at intervals as the process continued. The material was Swedish iron following Hooke's law (in tension) up to 13 tons per sq. in. and having a well-marked yield-point at 14.1 tons per sq. in. It was found that the material suffered no damage from repeated reversals of a stress of 5 tons per sq. in., but that when the greatest stress was raised to 7 tons per sq. in. incipient signs of fatigue began to be apparent after many reversals, though the piece was still intact after the number of reversals had reached three millions. With

a stress of 9 tons per sq. in., or more, repeated reversals brought about fracture. The first sign of fatigue as detected in the microscope was that slip lines began to appear on one or more of the crystals in the region of greatest stress: as the process went on these became more distinct and tended to broaden, and at length some of them developed into cracks which were identified as such because they did not disappear when the surface was repolished. Once a crack had formed it quickly spread, and finally the piece broke with a sharp fracture, showing practically no plastic change of form before rupture."*

Probably the most extensive series of experiments with rotating shafts were made at the Watertown Arsenal during a period of ten or more years prior to 1908. A synopsis of some of these experiments—a great many tests of hot-rolled open-hearth steel bars—is given in Table 3. It is interesting to note that in one case the fracture started from a slight indentation and in another case from a slight scratch on the surface of the shaft. With the exception of 0.73% carbon steel, the pulling tests showed well-marked yield points, either at the elastic limit or very slightly above. The results of pulling tests from ruptured bars agreed substantially with those of the auxiliary pulling tests, except for a slight decrease in elongation. The 1.09% carbon steel failed under alternating stresses of an intensity 51.9% of the elastic limit. Omitting this steel from consideration, on account of its being much higher in carbon than steel used for structures, the lowest intensity, relative to the elastic limit, at which the steel failed in these endurance tests was 61.4% of that limit. The extreme intensities of the alternating stresses recorded in these tests were computed for static conditions, in accordance with the theory of flexure; but, as this theory gives results which, when the elastic limit is not exceeded, are too low, and as the shafts were rotating 500 times per min., the actual extreme intensities and their ratios to the elastic limit were, probably, somewhat greater than are given in the table.

An extensive set of experiments was made "On Resistance of Iron and Steel to Reversals of Direct Stress" by Thomas Ernest Stanton and Leonard Bairstow.† The specimens they used were tested by direct instead of bending stress. Table 4 gives the results of the critical tests, that is, those in which rupture occurred at the least

* James Alfred Ewing, in Encyclopædia Britannica, 11th Ed., Vol. XXV, p. 1019.
† *Minutes of Proceedings*, Inst. C. E., Vol. CLXVI, Part 4, 1906.

TABLE 3.—ENDURANCE TESTS OF ROTATING SHAFTS TURNED TO 1.009 IN. DIAMETER, FROM 1¼-IN. HOT-ROLLED BARS OF OPEN-HEARTH STEEL OF VARIOUS GRADES, AS FOLLOWS:

	Percentages.					
Carbon	0.17	0.34	0.55	0.73	0.82	1.09
Manganese	0.57	0.65	0.75	0.64	0.36	0.39
Silicon	0.04	0.34	0.14	0.04	0.10	0.11

Length between supports, 33 in.,
Loaded over 4 in. length at middle,
Speed of rotation, 500 per min.

Compiled from Watertown Arsenal Reports for Years 1903-1908.

PULLING TESTS AUXILIARY TO ENDURANCE TESTS. GAUGED LENGTH, 4 IN.			Percentage of carbon.	ENDURANCE TESTS OF ROTATING SHAFTS.					
					ALTERNATE TENSION AND COMPRESSION.				
Test No.	Elastic limit, in pounds per square inch.	Tensile strength, in pounds per square inch.		Test No.	Maximum fiber stress, in pounds per square inch.	Percentage of elastic limit.*	Percentage of tensile strength.*	Number of rotations.	Result of test on shaft.
7 937	51 000*	68 000*	0.17	371	60 000	117.6	88.2	6 470	Ruptured.
			0.17	372	50 000	98.0	73.5	17 790	do.
			0.17	373	45 000	88.2	66.2	70 400	do.
			0.17	374	40 000	78.4	58.8	298 510	do.
			0.17	376	35 000	68.6	51.5	5 757 920	do.
			0.17	375	30 000	58.8	44.1	100 000 000	Not ruptured.
8 132	54 400*	85 000*	0.34	389	60 000	110.3	70.6	14 630	Ruptured.
8 133	54 200	84 200	0.34	390	50 000	91.9	58.8	69 350	do.
			0.34	395	45 000	82.7	52.9	166 360	do.
			0.34	412	40 000	73.5	47.1	68 667 320	do.†
7 938	57 000*	106 100*	0.55	377	60 000	105.3	56.6	12 490	Ruptured.
			0.55	378	50 000	87.7	47.1	93 160	do.
			0.55	379	45 000	78.9	42.4	166 240	do.
			0.55	380	40 000	70.2	37.7	455 350	do.
			0.55	381	35 000	61.4	33.0	900 720	do.
			0.55	383	30 000	52.6	28.3	75 006 000	Not ruptured.
8 135	66 000*	141 200*	0.73	391	60 000	90.9	42.5	55 390	Ruptured.
8 134	64 000	140 200	0.73	392	50 000	75.8	35.4	238 212	do.
			0.73	396	45 000	68.2	31.9	7 254 010	do.‡
			0.73	417	40 000	60.6	28.3	58 400 000	Not ruptured.
7 944	63 000*	142 250*	0.82	382	60 000	95.2	42.2	37 250	Ruptured.
			0.82	384	55 000	87.3	38.6	93 790	do.
			0.82	386	50 000	79.4	35.1	213 150	do.
			0.82	387	45 000	71.4	31.6	605 460	do.
			0.82	388	40 000	63.5	28.1	202 000 000	Not ruptured.
8 137	77 000*	132 800*	1.09	393	60 000	77.9	45.2	17 540	Ruptured.
8 136	75 400	132 000	1.09	394	50 000	64.9	37.7	61 030	do.
			1.09	403	45 000	58.4	33.9	182 700	do.
			1.09	404	40 000	51.9	30.1	433 380	do.
			1.09	405	35 000	45.5	26.4	175 280 000	Not ruptured.

* Designates elastic limit and tensile strength used in computing percentages.

† Bar ruptured midway between middle bearings, at a score mark made by head of screw which holds up middle bearing fixture.

‡ Eccentric fracture at fine line scratched on the surface of the bar which was used for locating the position of the micrometer beam.

intensity of stress, and those which endured the greatest number of reversals of stress without rupture. In these tests, as in those described by Ewing, microscopical examination during their progress showed that failure was caused by the gradual development of cracks.

The tests recorded in Tables 3 and 4 have a direct bearing on the strength of moving parts of machinery. As regards wrought-iron and steel structures, although the conditions are not precisely analogous, it may be inferred that a somewhat similar fatigue would occur, in those parts which are subjected to frequent alternations of stress, if certain limits less than the elastic limit were exceeded. In structural

TABLE 4.—Resistance of Iron and Steel to Reversals of Direct Stress.

Compiled from Stanton and Bairstow's paper in *Minutes of Proceedings*, Inst. C. E., Vol. CLXVI, 1906, Part IV.

Stresses were direct and produced by a periodic motion, about 800 cycles per minute, of a reciprocating mass by means of a connecting rod, the specimen under test forming the link between the reciprocating mass and the cross-head.

The ratio of tension to compression was: for Items 1 to 18, 1.4; for Items 19 to 22, 1.09; for Items 23 and 24, 0.92; for Items 25 to 27, 0.72.

Item 19 was one of a set which the authors believed to be subject to some secondary effect, to avoid which they made test pieces somewhat smaller for set including Items 20 to 22.

All stresses are in tons per square inch.

Materials used in experiments.	Chemical Analyses. Percentages of:					Tension Tests.		
	C.	Mn.	Si.	S.	P.	Primitive elastic limit.	Maximum load.	Total elongation in 8 in. Percentage.
Swedish Bessemer Steel No. 3	0.645	0.260	0.062	0.010	0.028	27.66	47.60	12.9
Swedish Bessemer Steel No. 2	0.446	0.370	0.058	0.012	0.028	25.00	43.75	17.0
Swedish Bessemer Steel No. 1	0.170	0.100	0.021	0.012	0.013	21.42	28.53	22.8
Piston-rod steel	0.446	0.470	0.063	0.044	0.067	19.62	43.85	18.1
Steel forging	0.336	0.560	0.072	0.021	0.026	12.94	29.47	16.6
Mild steel No. 2	0.331	0.680	0.086	0.056	0.066	14.30	28.30	24.6
" " No. 1	0.065	0.040	0.148	0.010	0.135	10.72	21.92	28.0
Swedish charcoal iron	0.039	Trace	Trace	0.000	0.018	12.93	19.60	33.8
Wrought iron No. 2	0.195	0.005	0.086	0.011	0.054	13.37	25.58	23.8
" " No. 1	0.029	0.070	0.127	0.024	0.219	14.28	23.76	27.0

TABLE 4.—(*Continued.*)

Item.	Material.	Maximum tension.	Maximum compression.	Maximum range of stress.	Ratio of tension to elastic limit.	Ratio of range of stress to elastic limit.	Number of reversals.	Result of test on specimen.
1..	Swedish Bessemer Steel No. 3.	18.95	13 54	32.49	0.685	1.175	204 000	Broken.
2..	do. No. 3.	18.28	13.05	31.33	0.661	1.133	1 561 000	Not broken.
3..	do. No. 2.	17.60	12.57	30.17	0.704	1.207	129 500	Broken.
4..	do. No. 2.	16.97	12.13	29.10	0.679	1.164	1 287 000	Not broken.
5..	do. No. 1.	15.63	11.17	26.80	0.730	1.251	1 001 000	Broken.
6..	do. No. 1.	14.96	10.69	25.65	0.698	1.197	2 148 000	Not broken.
7..	Piston Rod Steel.....	16.80	12.00	28.80	0.856	1.468	752 000	Broken.
8..	do.	16.26	11.62	27.88	0.829	1.421	3 409 000	Not broken.
9..	Steel Forging.....	12.31	8.79	21.10	0.951	1.631	886 300	Broken.
10..	do.	11.14	7.95	19.09	0.861	1.475	1 782 800	Not broken.
11..	Mild Steel No. 2.....	15.00	10.71	25.71	1.049	1.798	1 330 000	Broken.
12..	do. No. 2.....	13.59	9.70	23.29	0.950	1.629	2 000 000	Not broken.
13..	Mild Steel No. 1.....	11.05	7.90	18.95	1.031	1.768	1 055 300	Broken.
14..	Swedish Charcoal Iron.....	11.17	7.98	19.15	0.864	1.481	693 500	Broken.
15..	do.	10.77	7.70	18.47	0.833	1 428	1 360 000	Not broken.
16..	Wrought Iron No. 1.....	12.39	8.85	21.24	0.868	1.487	214 900	Broken.
17..	do. No. 1.....	11.78	8.42	20.20	0.825	1.415	1 116 300	Not broken.
18..	do. No. 2.....	11.26	8.04	19.30	0.842	1.444	904 800	Broken.
19..	Wrought Iron No. 2.....	9.28	8.51	17.79	0.694	1.331	942 000	Broken.
20..	Swedish Charcoal Iron	10.37	9.51	19.88	0.802	1.538	271 800	Broken.
21 .	do.	9.86	9.04	18.90	0.763	1.462	1 345 000	Not broken.
22..	Wrought Iron No. 2.....	10.09	9.26	19.35	0.755 Compression to elastic limit.	1.447	1 045 000	Not broken.
23..	Swedish Charcoal Iron.....	9.06	9.84	18.90	0.838	1.462	486 600	Broken.
24..	do.	8.46	9.19	17.65	0.711	1.365	1 029 800	Not broken.
25..	do.	7.69	10.69	18.38	0.827	1.422	523 500	Broken.
26..	do.	7.36	10.22	17.58	0.790	1.360	1 004 000	Not broken.
27..	Wrought Iron No. 2.....	8.10	11.25	19.35	0.841	1.447	1 045 000	Not broken.

members, however, as the frequency of the alternations is much less than in these tests, and as periods of rest are interspersed between alternations, it is fair to assume that the limits between which the stresses can alternate are greater than in the tests. It is a fair inference from the tests cited that stress of great intensity in tension or compression reduces the capacity of a structural member to resist immediately, without injury, stress of the opposite kind, but not to the extent indicated in Tables 3 and 4.

In the case of a member subject to alternate tension and compression of equal intensity, if the capacity of the steel to resist either stress were reduced to two-thirds of what it would be to resist one or the other without any alternations, the effect on the sectional area required would be equivalent to that of a 50% increase in the total stress in a member subject to only one kind of stress; and the ratio

of the range of stress (that is, tension plus compression) to the capacity of the metal when there are no alternations of stress to resist without injury either kind, would be twice two-thirds or 1.333. When this ratio is compared with the ratios of range of stress to elastic limit given in Table 4, in the light of the differences in conditions, the assumption from which it was derived seems reasonably safe. The proportion chosen for this tentative assumption was such as gives, when the alternate stresses are equal, some basis of comparison of Table 4 with the provision for alternating stresses given in the American Railway Engineering Association's General Specifications for Steel Railway Bridges. This provision, taken from the third edition (1910), Paragraph 22, is as follows:

"Members subject to alternate stresses of tension and compression shall be proportioned for the stresses giving the largest section. If the alternate stresses occur in succession during the passage of one train, as in stiff counters, each stress shall be increased by 50 per cent. of the smaller. The connections shall in all cases be proportioned for the sum of the stresses."

In certain conditions, some of the fatigue under alternating stresses within the elastic limit is temporary. Lord Kelvin and other experimenters have observed such a fatigue in vibrating wires. Wires which had been kept vibrating for several hours or days, within certain limits much below the elastic limit, came to a rest much sooner when left to themselves than when set in vibration after they had been at rest for several days and were then immediately left to themselves. This may have been due to initial internal stresses which, in combination with stresses from the external forces, over-strained small portions of the metal and made them viscous. The rapid vibration of the wires may then have prevented the viscous metal from solidifying, and gradually increased the fatigue. During a rest, the viscous portion of the metal would resolidify, and new internal stresses would be developed, as previously explained in discussing flexure.

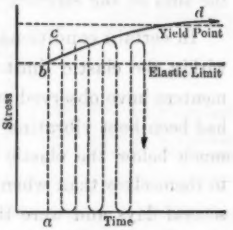
It is a matter of experience that, when there are bad faults in steel or wrought iron, failure may occur under stresses (which are repeated or continued) of computed intensity less than the elastic limit, with little prior warning in the way of deformation. It is conceivable that failure of steel and wrought iron from fatigue might similarly occur under repeated or continued stresses within the elastic limit,

even when the initial flaws are microscopic; but there is nothing in experience or tests to cause fear of such a result when the material is of good quality and the stresses are not reversed. Of course, failure may occur in compression from slenderness under stresses within the elastic limit, but this has nothing to do with fatigue.

The most thorough, scientific, and valuable experimental investigation of repeated stresses in steel and wrought iron was made by the late Professor Johann Bauschinger. In these experiments he used a remarkable measuring device which was sensitive to $\frac{1}{10\,000}$ mm. (about $\frac{1}{250\,000}$ in.), and the repetitions of stress ranged from 5 000 000 to 10 000 000, or more.

The results of this profound investigation by this noted scientist were published in 1886,* and were well epitomized by Professor W. C. Unwin in discussing Stanton and Bairstow's paper on reversals of stress, as follows:

"Bauschinger was the first to notice the remarkable variation of the position of the true limit of elasticity—not the yield-point, but the limit of proportionality—when a bar was subjected to straining actions. The important Paper which he wrote in 1885 dealt almost wholly with the effect of variations of tensile stress, and the point which Bauschinger fully made out was illustrated diagrammatically by Fig. 38.† Suppose a bar subjected to a stress varying between 0 and a limit *c* above the real primitive elastic limit. The successive applications of the stress were shown by the dotted line. The elastic limit initially at *b* would rise sometimes above the primitive yield-point as shown by the strong line *b d*. Now if *d* was above the applied stress *c*, no number of repetitions of stress would break the bar. But if *c* was above *d*, the point to which the elastic limit rose, then the bar would break after a certain number of repetitions of stress. That was for variation of stress in one direction, and he thought it was something more than a mere theory, and that the point had been completely made out."



(Unwin's Fig. 38)
FIG. 6.

It is evident that when the stresses are not reversed the elastic limit of steel and wrought iron in structures is the criterion of safety with respect to stress intensity, and that the real factor of safety in

* Communication No. 13, Testing Laboratory, School of Technology of Munich.
† Reproduced herein as Fig. 6.

such cases is the ratio of the elastic limit to the actual stress, instead of the ratio of the momentary ultimate strength to the actual stress.

For steel and wrought-iron railway bridges, when the actual stresses in members subjected to only one kind of stress are within the elastic limit, and when the actual stresses in members subjected to alternate stresses are within two-thirds of said limit, it is reasonable to assume that the life of these structures will depend on other considerations. Although experiments do not warrant the conclusion that steel and wrought iron can resist endless repetitions of stress to those limits, neither do they warrant the conclusion that these metals cannot do so; and both experiments and experience indicate that steel and wrought iron in bridges can resist to these limits for so long a period that the resistance may for practical purposes be considered permanent. This, of course, does not mean that a structure is necessarily safe if the computed stresses are within the supposed elastic limit (or two-thirds thereof if the stress alternates); for the real stresses may be greater than computed and the actual elastic limit less than supposed, and the compression members may be lacking in stiffness and the structure unstable.

The fatigue or weakening of a piece of steel or wrought iron which occurs when it is strained in tension and compression alternately beyond certain limits (which are often well within the elastic limit) makes it subject to sudden fracture, without much preliminary deformation; similarly, as in a piece in which the crystals or grains have united in regular crystalline formation without much mechanical interference from hot-forging, rolling, or pressing; hence such weakening is often called crystallizing, and by some is actually conceived to be a phenomenon in crystallogeny—an idea fostered by the fact that the crystals or grains of which steel and wrought iron are composed are displayed in sudden fractures but are more or less disguised in ordinary tests for quality by being drawn out before they are broken.

There is probably no change, in the structure of metals, from strain, so long as no part is over-strained. When metals are made partly viscous by over-straining and are then allowed to resolidify, the resolidified portion, of course, has a new structure. In lead, according to Ewing, severe over-straining is followed, even at atmospheric temperatures, by a protracted crystalline growth which results in the

formation of crystals which are relatively very large.* There is no evidence, however, that steel and wrought iron made viscous by over-straining resolidify in large crystals at atmospheric temperatures. Atmospheric temperatures are so far below the recalescence point of steel and iron that it seems reasonable to expect that the viscous metal at such temperatures will not resolidify in large crystals. There is, in fact, considerable to indicate that films of steel made viscous by over-straining will, after resolidification, be in an amorphous condition; that is, not crystallized, even in the minutest particles. It would seem that the solidification is, in the main, so rapid that the crystals do not have time in which to grow to discernible size.†

Launhardt's Fatigue Formula.

As has been shown, the elastic limit of structural steel and wrought iron can be raised by over-straining followed by a rest. Experiments by Wöhler during the period 1858-1870 indicate that it can also be raised when part of the stress is constant and part is constantly repeated, even without intervals of rest, and seem to indicate that the amount by which it can be raised increases with the proportion of constant stress. The phrase "seem to indicate" is used deliberately,

* Encyclopædia Britannica, 11th Ed., Vol. XXV, p. 1018.

† "About ten years ago [dating from May, 1911] the microscopic study of the structure and behavior of metal led to the discovery of the true nature of the operation of polishing. In this operation a true skin is formed over the polished surface. This skin gives unmistakable signs that it has passed through a state in which it must have possessed the perfect mobility of a liquid. In its final state it possesses distinctive qualities which differentiate its substance very clearly from that of the unaltered substance beneath it. It is, for instance, much harder and, even when formed on the face of a crystal on which the hardness varies in different directions, its hardness is the same in all directions. * * * The discovery that layers of a solid many hundreds of molecules in thickness can have a mobility of the liquid state conferred on them by purely mechanical movement, opened up a new field of inquiry into internal structure of metals which have been hardened by cold working. As a result of this inquiry a theory of the hard and soft states was suggested. According to this theory, hardening results from the formation at all the internal surfaces of slip or shear of mobile layers similar to those produced on the outer surface by polishing. These layers only retain their mobility for a very brief period and then solidify in a vitreous amorphous state, thus cementing material at all surfaces of slip or shear throughout the mass. * * *

"The ductile metals, when pure, do not pass into a vitreous amorphous state by cooling, because their molecules retain the power of crystalline rearrangement over a very long range of temperature below the solidifying point. Gold solidifies at 1080° and its minimum crystallizing temperature is somewhere about 230°. There is thus a range of about 800° over which crystallization can occur. It is obvious that no ordinary method of chilling could carry a mass of metal over this long range so quickly that it could not crystallize. Though the arrest of crystallization by chilling the molten metal is practically impossible, this arrest can and does occur in the thin layers of mobile molecules which are produced by mechanical flow. The evidence from observations which have extended over nearly ten years has made it certain that metals can exist in the vitreous amorphous form, and that the recrystallization which occurs when the point of thermal instability is reached is strictly analogous to the similar phenomenon which occurs when other well-known vitreous substances are kept at a suitable temperature in the presence of crystalline nucleus." From "1911 May Lecture", by G. T. Beilby, LL.D., F. R. S. (Member of Council) on "The Hard and Soft State in Metals", published in the *Journal* of the Institute of Metals, Vol. VI. pp. 5, 6, and 9.

for the reasons that the experiments in which the stress was produced by direct pull were meager and inconclusive, and the nominal stresses recorded in the extensive bending tests were computed by the ordinary theory of flexure, which is not even approximately correct when the elastic limit is exceeded.

Following Wöhler's experiments, Launhardt devised what is known as a fatigue formula, which is supposed to indicate the farthest point to which the stresses can be carried just to avoid failures from fatigue under various proportions of constant and constantly repeated stresses.

Launhardt's formula, as given by Weyrauch for wrought iron, is

$$a = u\left(1 + \frac{1}{2}\frac{\text{Minimum stress}}{\text{Maximum stress}}\right) \quad \ldots\ldots\ldots\ldots (1)$$

where

a = ultimate working strength per unit of section under the conditions assumed,

u = ultimate strength per unit of section for any number of repetitions of load.

In Wöhler's tests, a bar repeatedly pulled from 0 to 320 centners per square zoll, finally ruptured on the 10 141 645th application of the stress.

320 centners per square zoll is a little more than 30 000 lb. per sq. in.*

Adopting the square inch and the pound as units, and taking the value of u as 30 000, the Weyrauch-Launhardt formula for wrought iron becomes:

$$a = 30\,000\left(1 + \frac{1}{2}\frac{\text{Minimum stress}}{\text{Maximum stress}}\right) \quad \ldots\ldots\ldots\ldots (2)$$

According to this formula, the greatest stress per square inch to which wrought iron can be subjected without permanent fatigue varies from 30 000 lb., in cases where all the stress is constantly repeated, to 45 000 lb., in cases where it is all constant.

As even 30 000 lb. is above the elastic limit of ordinary wrought iron, such metal would have to have its elastic limit raised somewhat more than 50% by over-straining before it could stand permanently 45 000 lb. per sq. in.

It is not reasonable to apply this and similar formulas to the design of bridges, because two of the fundamental considerations on which such formulas rest are not realized in these structures: the

* According to Bauschinger (*loc. cit.*, p. 44), the centner per square zoll in which Wöhler gives his results in equivalent to 6.837 kg. per sq. cm., or 972 lb. per sq. in.

stresses are not constantly repeated without intervals of rest; and the permanent strength cannot possibly be increased, in non-adjustable tension and compression members, far above the primitive elastic limit, for the reason that the change in length of such members when strained beyond the yield point is so great as to be ruinous. After once being strained a little beyond the yield point, there would be no repeated stresses. It is quite probable, however, that, at the expense of some undesirable though not ruinous distortion of a structure, its tension members and very short compression members could have their elastic limit and practically permanent strength raised nearly to the yield point.

A considerable number of American engineers use a formula, in appearance greatly resembling Launhardt's, which evaluates the permissible working stress per square inch with quantities intended to give a constant factor of safety, with respect to the capacity of the metal just to avoid permanent fatigue, and at the same time allow for impact. As an illustration, the following formula is quoted, from the specifications of a large railway system, for permissible tension per square inch.

$$\text{Tension, rolled bars, } = 7\,500\left(1+\frac{\text{Minimum stress}}{\text{Maximum stress}}\right) \ldots\ldots (3)$$

In explaining this formula, the capacity of the metal just to avoid permanent fatigue is assumed to be a function of the maximum permissible stress (from 7 500 to 15 000 lb. per sq. in.), instead of a function of the maximum permanent strength (from 30 000 to 45 000 lb. per sq. in., according to the minimum); a modification for which, as far as the experiments or theories regarding fatigue are concerned, there is no warrant.

Some engineers, who realize that Formula (3) is wholly unsound with respect to the possibilities of fatigue in bridges, continue to use it, on the ground that it makes a good provision for impact. Those who advocate it solely as a provision for impact should use an equivalent method of making such provision, which clearly defines their position, is much more convenient, and brings precisely the same sectional area, as follows:

$$\text{Impact} = \text{live load}\left(\frac{\text{Maximum stress}}{\text{Maximum stress} + \text{minimum stress}}\right) \ldots (4)$$

Permissible tension in rolled bars = 15 000 lb. per sq. in.

The chief objection to formulas such as (3) and (4) is that, for long-span bridges, they make the provision for dead load relatively too small as compared with the provision for live load; which may be dangerous when the permissible unit stress is high and the dead load predominates.

In the case of bending stresses, either by themselves or in combination with direct tension or compression, there is some opportunity for enhancement of the elastic limit, without serious distortion, under constant stresses or repeated stresses in one direction, especially in solid members such as pins; but, in the case of bending stresses in combination with direct compression, the opportunity is quite limited.

Acknowledgment is made of valuable suggestions from C. E. Chase, Jun. Am. Soc. C. E., who kindly made a critical review of the paper in its preliminary form.

DISCUSSION

Mr. Waddell.

J. A. L. WADDELL,* M. AM. SOC. C. E. (by letter).—This paper is exceedingly interesting and valuable, for these two reasons:

First.—The author has given to the Engineering Profession in concise form a history and a complete explanation of the phenomenon known as "the fatigue of metals".

Second.—He has shown that, as far as modern bridge designing is concerned, the fatigue of steel is a matter that can, with the utmost propriety, be totally ignored.

Ever since bridge designing began to become truly a science—about a quarter of a century ago—the writer has maintained that the fatigue of metal has no application whatsoever in the proper proportioning of bridge members. Judging from the character of modern bridge specifications, most of the authorities now agree with him; but a few still use formulas which are based, either directly or indirectly, on the fatigue theory; and intelligent laymen are prone to talk about the crystallization of steel in bridges—something which does not exist and never did exist, even in the old days of inadequately proportioned railroad structures. Consequently, it is very gratifying to the writer to see how effectively Mr. Prichard has exposed the fallacy which for many years gave to bridge computers so much extra labor which was totally unnecessary and valueless.

For non-reversing stresses, Mr. Prichard points out that, even with wrought iron, under the worst possible circumstances, the actual intensity of working stress must be as great as 30 000 lb. in order to produce any fatigue that will ultimately result in rupture; and as the weakest bridge steel is fully 20% stronger than wrought iron, the corresponding limit for that metal would be 36 000 lb. Now, as modern bridge specifications stress medium-carbon steel up to only 16 000 lb. per sq. in. for the usual loadings when the effect of impact is included, it is evident that the factor of safety is about 2.25. It is true that, when the greatest possible wind-load stresses are combined with the maximum live, impact, and dead-load stresses, it is permissible to stress the metal 30% higher, or to 20 800 lb. per sq. in., which would reduce the factor of safety to 1.73; but this is practically an impossible loading; hence it may be concluded that, as far as fatigue without stress reversal is concerned, modern bridges have a factor of safety of about 2. This is used so as to cover the possibility of small imperfections existing in the metal; and it is an ample allowance for that and all other considerations.

For the case of reversing stresses, however, it is not quite so evident as it is in the case of non-reversing stresses that the effect of

* Kansas City, Mo.

Mr. Waddell.

fatigue can be entirely ignored; hence a simple computation will be made to prove it.

In the first place, it should be stated that for ordinary bridges the members subjected to alternating stresses are not affected by wind, hence the excess percentage for intensities with wind-load combinations need not be considered. The writer's latest specifications for the effect of reversing stresses provide that when the reversion comes from a single live loading, the sectional area of any piece is to be computed by adding to the maximum tension 75% of the maximum compression and dividing the sum by 16 000; also, by adding to the maximum compression 75% of the maximum tension and dividing the sum by $16\,000 - a\frac{l}{r}$, where a is equal to 60 for riveted construction or 80 for pin-connected construction. In case the reversion is produced by some other live loadings, the same method is to be followed, except that 50% instead of 75% is to be added. The larger of the two areas thus obtained is, of course, the one to be adopted.

The worst case of reversion that can be found in ordinary bridge designing is that of the web diagonal at the foot of the middle vertical post in a very long-span, simple-truss bridge having an even number of double panels. If it is assumed that the live-load tensile stress on the diagonal, including impact, is 700 000 lb., the dead-load tension thereon will be about 100 000 lb., and the live-load compression (including impact) about 570 000 lb.

Proportioning for tension, we have:

$$A = \frac{750\,000 + 100\,000 + 0.75 \times 570\,000}{16\,000} = 76.7 \text{ sq. in.}$$

Adding 20% for rivet-holes makes 92 sq. in.

Proportioning for compression, assuming that $\frac{l}{r} = 50$, and that the truss is pin-connected, we have:

$$A = \frac{570\,000 - 100\,000 + 0.75 \times 700\,000}{12\,000} = 83 \text{ sq. in.}$$

It is seen, therefore, that it is the tension which counts; and, consequently, the maximum actual intensity on the net section will be

$$\frac{700\,000 + 100\,000}{76.7} = 10\,400 \text{ lb., nearly.}$$

The elastic limit for medium bridge steel is 35 000 lb. per sq. in., and, as the worst case of percentage for fatigue of such steel given in Tables 3 and 4 may be taken as not exceeding 60, the safe limit for reversion would be $35\,000 \times 0.6 = 21\,000$ lb., or almost exactly twice

Mr. Waddell. the actual intensity just found for the most unfavorable condition of stress reversion.

From these rough figures it is safe to conclude that, as far as the old bugbear of fatigue of metal is concerned, modern steel bridges will have a factor of safety of 2 after the effects of impact from live load have been included.

Mr. Prichard's paper certainly ought to put an effective stop to any further talk among engineers concerning fatigue of metal in steel structures, and to expressions of dread from educated laymen about the danger to the traveling public from crystallization of metal in bridges. If such a result is attained, Mr. Prichard will truly deserve the hearty thanks, not only of the Engineering Profession, but also of people in general, for his exhaustive and masterly memoir.

Mr. Seaman. HENRY B. SEAMAN,* M. AM. SOC. C. E. (by letter).—In referring to the use of the Launhardt formula for bridge design, Mr. Prichard states:

"It is not reasonable to apply this and similar formulas to the design of bridges, because two of the fundamental considerations on which such formulas rest are not realized in these structures, * * *."

The writer, however, would call attention to the fact that objections to the use of the Launhardt formula rest on much broader grounds, namely, the formula itself is not justified, either in its manner of deduction, or in the tests on which it rests. This was discussed fully before the Society in 1899.†

Mr. Turner. C. A. P. TURNER,‡ M. AM. SOC. C. E. (by letter).—This paper again brings up for discussion the maximum and minimum formulas which have largely disappeared from bridge specifications to-day. These formulas are based on the assumption that there is some direct relation between detrimental effects of high and low stresses, some of these stresses being below the so-called elastic limit of the metal. The elastic limit is determined commercially by the drop of the beam; but, in more precise tests, the extensometer is used and the set noted. The degree of elasticity thus determined, however, even when there is no set or deformation, does not determine the true elastic limit, because some of the work applied to the bar in stretching it may not be given back in its return to its original form during removal of the load, so that it would take a very delicate apparatus indeed to determine what is going on within the bar. In general, it is known that every set or permanent distortion is accompanied by heat. It is

* New York City.

† "The Launhardt Formula, and Railroad Bridge Specifications", *Transactions*, Am. Soc. C. E., Vol. XLI, p. 140.

‡ Minneapolis, Minn.

known that the bar grows colder as it is stretched, until some set takes place. Mr. Turner.

By delicate thermo-electric means,* it is possible to determine what is going on within a bar and how far that bar may be stretched without the evolution of heat accompanying either deformation or disturbance of the internal structure. These limits, as determined approximately by thermo-electric measurement, seem to be the same as those within which Bauschinger concludes that, unless there are flaws, range of stress improves the elasticity, ductility, and tenacity of the specimen. In other words, within this range of true elasticity, there is no such thing as fatigue, which later may be defined as a change in the structure of the bar brought about by excessive stress. If the viewpoint be in fact that just outlined, then fatigue formulas have no place in the proper design of bridge structures in which the intention is to keep stresses within such well-defined limits that overstrain does not occur. Over-confidence in the logical foundation of maximum and minimum formulas seems to have been responsible, in part at least, for the high stresses allowed in portions of the ill-fated Quebec Bridge. More uniform working stresses, with a slight improvement in the make-up of the sections, would undoubtedly have prevented this unfortunate failure. The acceptance by some of maximum and minimum formulas seems to have arisen, not from the teaching of experience in bridge construction, but rather on account of their plausibility from an algebraic standpoint and a disregard of any precise distinction between true and approximate values of the elastic limit.

T. D. Lynch,† Assoc. M. Am. Soc. C. E. (by letter).—The writer's experience has shown that very few engineers realize how low the true elastic limit really is in untreated or annealed structural steel. Mr. Lynch.

Plates having a tensile strength of more than 60 000 lb. per sq. in. may, and often do, have a true elastic limit of 15 000 lb. per sq. in. or less, and the fact that it has been customary to use a large factor of safety is the only excuse many structures have for not falling.

As engineering progresses toward economical production, and designers work to closer and closer limits, it is necessary to study more carefully the real characteristics of the material used.

This has been found to be true in the manufacture of heavy machinery, and structural engineers will find it equally true in making safe their great constructions of bridges and buildings. Those who are working to the "yield point" as taken by the drop of beam will find on final analysis that it means practically nothing unless used as indicative of the material's real value as having been determined by special tests to represent each class of material—untreated, heat-treated, or annealed.

* *Transactions*, Am. Soc. C. E., Vol. XLVIII, p. 140.

† Wilkinsburg, Pa.

Mr. Lynch. The writer is pleased to refer any member who may wish to examine this subject further to two papers* presented by him before the American Society for Testing Materials.

There seem to be two fundamentals that must not be overlooked, although in many cases research work may be necessary to determine them:

First.—Study stresses to know what they are.

Second.—Study the characteristics of the steel to be used; determine its true elastic limit; and keep below it.

These two fundamentals can but insure safety of construction.

Mr. Howard. JAMES E. HOWARD,† ESQ. (by letter).—Mr. Prichard presents a paper on the physics of iron and steel, which is of exceptional interest; its scope, if amplified, would constitute a treatise on the fundamental properties of structural materials.

It can hardly have escaped notice that a tendency has displayed itself recently to overlook some of the basic and underlying features which govern the physical properties of metals; and, in the examination of iron and steel, tests are multiplied, giving them considerable diversity without having scientific ends in view, and not attaining results of direct application to the conditions which prevail in the use of constructive materials.

On account of the attention which is given to the principles on which engineering practice in the use of steel should rest, and other reasons which are equally conspicuous, this paper possesses interest of unusual degree.

In the different places in which steels are used, the requirements vary, and metal suitable to meet these varied needs must be regulated according to the demands which are present. The properties of the materials should bear a definite relation to service requirements. This calls for a proper selection of metal in each case and a careful discrimination as to what constitutes the important factors, in order to promote durability under working conditions.

There are limitations in respect to the ability of all grades of steel to endure stresses. Steels of different composition have, in their primitive annealed states, definite limits of endurance, the maximum utilization of which is influenced by the manner in which they are loaded. In an unannealed state, or after having been subjected to heat or mechanical treatment, their properties are materially modified. It is desirable to ascertain the influence which different states of the steel have on its endurance for varying conditions of service. The

* "The Use of the Extensometer in Commercial Work", *Proceedings*, Am. Soc. for Testing Materials, Vol. VIII, 1908; and "Elastic Limit", *Loc. cit.*, Vol. XV, Part II, 1915.

† Washington, D. C.

Mr. Howard.

initial properties easily admit of modification, which the tests of the metal clearly show; nevertheless, it is not always certain what ultimate or permanent advantages accrue from the possession of these modified properties. This information must be acquired through special experimental inquiry or as the result of long exposure under service conditions.

It is desirable to ascertain what features, in the initial properties, are indexical of those which contribute toward final durability.

Establishing the value of the elastic limit is early brought to notice when determining the physical properties of steel. It would seem that very little difficulty should be encountered in agreeing on what constitutes this factor, whether for research or for commercial purposes. Experience with metals has been ample to establish the fact, at least within limits of practical importance, that the modulus of elasticity of steel has a constant value within the limits of over-straining forces, that is, until a decided permanent set has been given the metal.

This being the case, the stress-strain curve is necessarily a straight line up to the load which causes a permanent set. The departure of the curve from a straight line, therefore, must indicate that a permanent set has been given the steel. It does not appear necessary to release the load during testing to confirm such an indication.

In a general way, these remarks comprise the principal statement which need be made in reference to defining the elastic limit of steel. Whether there is or is not a well-defined elastic limit in a given piece of steel, due to composition or treatment, and what degree of responsibility attaches to the manipulation of the test in obscuring that limit, are matters to be considered, but do not constitute a part of the main question.

Sharpness of definition of the elastic limit depends on the grade of steel, and on its state of initial strain. Concerning initial strains, they are affected by the manipulation of the steel and by incidents connected with its manufacture.

In the milder grades, jogs appear in the stress-strain curves, the metal displaying considerable elongation without increase of load, or under slightly diminished stresses. Jogs also appear when the metal is moderately heated, and disappear when higher temperatures are reached. Reversal of stress, from compression to tension—each being an over-straining load—removes the tendency to display a jog in the curve just beyond the elastic limit. The harder steels do not display jogs.

Initial strains obscure the sharpness of the elastic limit and tend to render its position vague. Initial strains may be acquired by a rapid rate of cooling during manufacture, and may result also from subsequent mechanical treatment, that is, from cold-working. Initial strains

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will be of two kinds, tension and compression, in order to maintain a state of equilibrium. The magnitude of one or both may at times reach and coincide with the elastic limit in those parts of the bar in which they exist.

An external force required to cause a permanent set will be the difference between the real elastic limit of the steel and the initial strain present which has the same algebraic sign as the external force. If internal initial strains coincide with the elastic limit, then any external force, however small, will cause a permanent set. Initial strains vary in their intensity in different parts of the cross-section of the bar, whence it follows that an external force gradually increased causes a progressive development of permanent sets. Local sets gradually extend to other parts of the cross-section, as external loads are increased, until the metal is in a state of over-strain throughout.

All rolled structural shapes acquire initial strains during fabrication, and these shapes necessarily display early local sets when loaded either by tension or by compression. The influence of such strains is particularly felt in tests on the ultimate strength of compression members. Initial strains, corresponding to many thousand pounds per square inch, residing in the component parts of built-up columns, and not given due consideration, necessarily tend to render futile all efforts to establish formulas for long columns.

Until loads are reached which cause sets to appear, the rate of extension or compression is not affected by the presence of initial strains in the steel. If the working stresses are low, the initial strains will pass unnoticed in structural members. It can hardly be said that the metal has no elastic limit when perchance initial strains exist in such degree that local sets are displayed under the earliest increments of load.

The effects of eccentricity in loading in the testing machine are similar to those caused by initial strains, in respect to the development of early sets. Relative to this matter, a prolific source of error is recognized which attaches to the manipulation of tests. Extensometer readings, of which the following are recent examples, showed measurements on three elements of a test piece, 120° apart, in which the stresses corresponding to the strains reported were 12 000, 15 000, and 18 000 lb. per sq. in., respectively. That is, the strain on one element was one and one-half times that on another. Early and progressive sets must necessarily have been developed on the side of the test piece which was strained so much in excess of that of the shorter side.

The modulus of elasticity is disturbed in over-strained steel, and temporarily has a lower value than normal. Temporarily, at least, it also has a variable value in over-strained steel. The reduction in value, however, is not of so pronounced a degree as to obscure the limit at which permanent sets are first acquired.

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A number of terms have been proposed in current technical literature to apply to that portion of the stress-strain curve which is in the vicinity of the elastic limit. It appears that a clear conception is not held concerning the underlying causes which affect this portion of the curve, and efforts directed toward increasing the terminology pertaining thereto seem to confuse rather than clarify the subject matter.

The importance which attaches to the accurate determination of the elastic limit, and the primitive state of the steel, depends somewhat on the uses to which it is to be put. In direct tension, service stresses range from a few thousand pounds up to (or exceeding) 150 000 lb. per sq. in. The latter stress is applied to steel in the form of hard-drawn wire. In most compression members the elastic limit practically fixes the maximum stress which the material will endure and maintain its shape without impairing the integrity of the structure of which the member forms a part.

For the purpose of enduring alternate stresses of tension and compression, the most satisfactory treatment of steel is one of some obscurity. Experience has demonstrated the desirability of limiting the bending loads in such cases to unit stresses of moderate degree. The problem of furnishing a steel of maximum durability for sustaining repeated alternate stresses has hardly been solved. The primitive properties of steel may be made to show decided increase in values by suitable regulation of its chemical composition, and also by heat and mechanical treatment. To establish whether these high, induced values are of permanent advantage is a matter which must be determined by exposure to different conditions of service, long continued.

There are certain situations in which complicated conditions exist. Steel rails are exposed to the direct bending stresses of train loads, which cause alternate stresses of tension and compression according to the position of the wheels with reference to given places on the rails, in addition to which there are internal strains at the top of the head and on the gauge side, due to the cold-rolling action of the wheels, and these strains augment cooling strains which are introduced in the rails in the process of manufacture.

The influence of primitive toughness in the metal and its advantages in steels when exposed to a large number of repeated alternate stresses of moderate degree is not apparent. Under such conditions all grades of steel may be ruptured without the display of appreciable elongation or contraction of area. Toughness of the metal is available for a time in resisting deforming loads, but its display is not witnessed in cases of repeated stresses, long continued. Some evidence exists which attaches importance to the tensile strength of the steel under these circumstances. Steels exposed to repeated alternate stresses at a blue heat have shown greater endurance than when tested cold. This in-

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crease in endurance takes place in the zone of temperature in which the tensile strength of the metal is much above that at atmospheric temperatures. In relation to this matter it should be remarked that the elastic limit is then somewhat lower than at ordinary temperatures of testing.

Slip bands in the micro-structure of steel have been pointed out as precursors to or attendant phenomena to rupture by repeated stresses. Microscopic examination has not in some cases furnished detectable changes in structure. In shafts which were ruptured by repeated stresses which strained the steel by amounts not exceeding one and one-third thousandths of an inch per inch, that is, up to 40 000 lb. per sq. in. fiber stress, rupture being reached after 150 000 000 repetitions, no observable change in structure was revealed. Still, it does not appear unlikely that manifestations of a macroscopic and microscopic order, witnessed in cases of pronounced deformation, may in kind be similar to those more minute changes which attend the rupture of steel under long-continued stresses of moderate degree. It would greatly aid in the matter of choosing the most suitable composition and treatment of steels if the phases through which rupture was reached were better known. It is a matter of no little regret that the research laboratories of America are not engaged in advance inquiry on these fundamental features which offer so much encouragement in their study.

In closing it may be remarked that Poisson's ratio, as found in tests conducted at the Watertown Arsenal, furnished a value for steel between one-third and one-fourth; to be more exact, the value appeared to be almost midway between these fractions.

Mr. Moore.

H. F. MOORE,* ESQ. (by letter).—This paper is an excellent discussion of the general action of iron and steel under stress, and should serve to place proper emphasis on the study of the physical behavior of the material in structures.

Mr. Prichard advocates the use of the term "elastic limit" to define the limit of proportionality of stress to strain. Some standard use for this term is highly desirable. At present "elastic limit" is used by some engineers to indicate the limit of proportionality of stress to strain, by other engineers to indicate the yield point of material, and by still others to indicate the stress at which permanent deformation of the material first appears. The last-named use of the term is recommended in the tentative standards of the American Society for Testing Materials, and is followed by most foreign writers. Some standard nomenclature is highly desirable, and though, for static conditions, the limit of proportionality could well be called the elastic limit, for conditions involving fatigue under repeated loads "permanent set" seems more significant than deviation from Hooke's law.

* Professor of Eng. Materials, Univ. of Illinois, Urbana, Ill.

Mr. Moore.

The writer believes that it is doubtful if the elasticity and proportionality of steel are perfect under any conditions, and would refer to the conclusions of Professor E. R. Hedrick, of the University of Missouri.*

Mr. Prichard quotes the test results of Hancock as a proof of the maximum shear theory for the failure of ductile materials. If the maximum shear theory holds for all conditions, the shearing strength of a material must be not greater than one-half the tensile or the compressive strength. Hancock's tests showed that, in general, the shearing strength of steel was greater than one-half the tensile strength, and the agreement of his "elliptical" formula with test results furnishes an empirical relation for various combinations of shearing stress with direct stress, rather than a confirmation of the maximum shear theory. Recent test results by Dr. Becker† indicate that for combined stress, in which the shearing stress is relatively low, the strain in steel is the determining factor for strength, and that, for steel, if the shearing stress is more than about 0.6 of the direct stress, the shearing stress governs. Dr. Becker shows that the results of Guest, Scoble, Hancock, and Mason, as well as those of his own tests, are in better accord with this dual law of strength than with any single law proposed for combined stress. Relative to this matter, the results of Matsumura and Hamabe‡ for cast iron are of interest. The results of their tests indicate that for cast iron the maximum strain theory holds.

The writer agrees with Mr. Prichard's conclusions as to fatigue of materials, but wishes to emphasize certain phases of fatigue phenomena. Attention is called to his statement: "There is nothing in experience or tests to cause fear of such a result [fatigue failure] when the material is of good quality and the stresses are not reversed". This is probably true for structural members for which the repetitions of stress number but a few million, but it would not necessarily be true for machine members in which the numbers of repetitions may number hundreds of millions.

The writer heartily agrees with the author's statement:

"Although experiments do not warrant the conclusion that steel and wrought iron can resist endless repetitions of stress to those limits [two-thirds of the statically determined elastic limit], neither do they warrant the conclusion that these metals cannot do so."

If experiments do not prove conclusively that materials can resist an infinite number of repetitions of stress, it would seem that safe

* "A Generalized Form of Hooke's Law", *Engineering News*, Vol. 74, p. 542, September 16th, 1915.

† Embodied in *Bulletin No. 85*, University of Illinois Engineering Experiment Station.

‡ *Memoirs* of the College of Engineering, Kyoto Imperial University, February, 1915.

Mr. Moore.

practice would dictate methods of computation for fatigue based on the supposition that destructive effects similar to those set up under high stress are set up with diminished intensity under low stress, and that it should be recognized that structures which could resist a few million repetitions of a given stress might fail under billions of repetitions of the same stress. This does not in any way call in question the author's statement:

"* * * both experiments and experience indicate that steel and wrought iron in bridges can resist to these limits for so long a period that the resistance may for practical purposes be considered permanent."

For the busiest steam railroad bridges, the number of repetitions in a normal "lifetime" for the bridge amounts to a million or two, and in most members the stress is not reversed. In other words, the criterion of static strength governs the design of bridges and buildings, rather than the criterion of fatigue strength. For machine members, and for structural members carrying reversed stress, the fatigue strength would be the governing factor in many cases.

Professor Basquin, of Northwestern University, has pointed out* that, for a considerable range of stress, the results of repeated stress tests of metals are well represented by an exponential equation:

$$S = \frac{C}{R^n}, \text{ or } \log. S = \log. C - n \log. R, \text{ in which}$$

S = fiber stress necessary to cause failure;
R = number of repetitions of stress necessary to cause failure; and
C and n are experimentally determined constants.

For the past year Mr. F. B. Seely, of the University of Illinois, and the writer have been engaged in an investigation of test data for repeated stress tests. Some tests have been made in the University of Illinois Materials Testing Laboratory, and data of other tests have been studied. As a result of this investigation the following modification of Basquin's formula is proposed:

$$S = \frac{B}{(1 - Q)\, N^{0.125}},$$

or $\log. S = \log. B - \log. (1 - Q) - 0.125 \log. N$, in which

S = fiber stress necessary to cause failure;
N = corresponding number of repetitions of stress (the "life" of the piece);
Q = ratio of minimum stress applied to maximum stress
($Q = 0$ for load varying from zero to a maximum,
$Q = -1.0$ for a completely reversed load);

* "The Exponential Law of Endurance Tests", *Proceedings*, Am. Soc. for Testing Materials, Vol. X, 1910, p. 625.

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B is an experimentally determined constant, approximate values of which are given in Table 5.

TABLE 5.

Material.	Value of B.
Structural steel and wrought iron	250 000
Steel, 0.45% carbon	350 000
Cold-rolled steel shafting	400 000
Tempered spring steel	600 000 to 800 000

It may be noted that these values are larger than those given by Mr. Seely and the writer in their paper before the 1915 meeting of the American Society for Testing Materials; the values in Table 5 are given as a revision of the earlier ones, and, in the opinion of Mr. Seely and the writer, are more reliable. An error in the method of deducing the value of B from test data has been corrected.

If the foregoing formula is applied to the determination of stresses in structural members for which the stress is not reversed (for structures, N is rarely more than 1 000 000 or 2 000 000 in the life of the structure), the value of S to cause fatigue failure, in practically all cases, will be found to be greater than the yield point strength of the material; hence static strength governs the design. This is in accord with the conclusions reached by Mr. Prichard. If, however, the stresses in machine members are investigated or those in oft-stressed structural members in which the stress is reversed, fatigue strength, in most cases, will be found to govern the design.

Different criteria of strength govern static strength and fatigue strength, respectively, and in the design of any structure or machine both static strength and fatigue strength should be considered.

Mr. Speller.

F. N. SPELLER, ESQ.* (by letter).—In reference to the last paragraph on page 1440, it is not often that the irregularities in elasticity of steel are referred to in papers of this kind. It is well known to most of those who are engaged in the physical testing of steel products that the true elastic limit of the metal is usually not developed on the first application of stress. A slight permanent set is generally found far below the true elastic limit. This seems to indicate that the stress on first loading is not always taken up equally by the grains and that there is, so to speak, some granular slack which must be taken in and the structure adjusted to the load until the stress is equally distributed in the section. The cause of this phenomenon is probably to be found in the irregular size and shape of the crystalline grains which constitute the normal structure of all metals. On the effect of straining of metals, Professor Sauveur makes the following suggestive statement†:

* Metallurgical Engr., National Tube Co., Pittsburgh, Pa.

† "The Metallography and Heat Treatment of Iron and Steel", 1916 ed., p. 93.

Mr. Speller.

"It is seen that, contrary to the general belief, the metals remain crystalline after the severest strain, and that the flow or plastic strain in metals is not a homogeneous shear such as occurs in the flow of viscous fluids, but is the result of a limited number of separate slips, the crystalline elements themselves undergoing no deformation."

The necessity for the adjustment of the structure of the metal by a preliminary stress applied below the elastic limit is recognized in some of the more modern specifications. For example, The Interstate Commerce Commission's "Shipping Container Specifications No. 3", for high-pressure cylinders, contains the following clause:

"Each finished and annealed cylinder must be subjected to a hydrostatic test of not less than 3 000 lb. per sq. in. in a water jacket, or other apparatus of suitable form, to furnish reliable data. The permanent volumetric expansion must not exceed 10% of the whole volumetric expansion at this pressure."

In "Specification No. 4" of the Interstate Commerce Commission (ammonia cylinders) a pressure of 800 lb. preceding the official test of 1 000 lb. is permitted. It would be better, in the writer's opinion, to make this preliminary test at a pressure more nearly approaching, but not exceeding, the official test.

The physical properties of steel are also temporarily altered by the mechanical machining of test pieces. It has become common practice to allow test pieces to "rest" for a day or two before pulling, in order to develop the true elongation and elastic limit. An increase of 2% elongation in 8 in. may be obtained after eliminating surface strains by this "rest cure", which is simply a form of slow annealing.

Mr. McCulloch.

J. A. McCULLOCH,* Esq. (by letter).—The writer has seldom had the privilege of discussing a paper which accords so nearly with his own conclusions.

At times he has been inclined to urge the restriction of the term "viscosity" to that measure of internal resistance which increases in direct ratio to the velocity of relative motion. It is a property of all fluids, by virtue of which they resist motion produced by shearing force, and, necessarily, it ceases with the cessation of motion; that is, viscosity offers no static resistance. For limpid liquids, something is known of the laws of this resistance to shearing motion. Little is known of its laws where substances offer also (at the same time) elastic resistance.

However, all know well that the restricted dimensional meaning of this term is violated in the ordinary measurements of the viscosity of water and the thicker liquids. Moreover, its present use, as describing the plastic flow of ductile metals, is justified by the best authorities, for Lord Kelvin says: "The term plasticity implies no more than does viscosity".

* McKeesport, Pa.

Mr. McCulloch.

The idea of a solid solution is difficult to comprehend, and does not at first appeal to our understanding, but the idea that many other substances are true fluids, even though they appear to be so solid as to be brittle to light blows, is equally difficult to grasp. Familiar examples are pitch and sealing wax. With elastic material, the case is even more difficult, because one finds it hard to conceive that the reversal of motion or lessening of stress changes the substance from liquid to solid. However, it is known that the conception of co-existing elastic and viscous actions has not led to experimentation which has yielded a satisfactory solution of the problem.* Difficulty in comprehending an idea is no refutation.

Misconception of the exhibition of elastic properties appears to be rather the rule than the exception. It may be due to false or partial education, or to the rush to train engineers in brief time, or to feeding "canned knowledge" in which no distinction is made between yield on primary loading and the recovery when the first load is released. For example, Pender gives:† "Ability to return to its original form after deformation is called elasticity"; and Kent states‡ that within the elastic limit, "* * * for such materials [iron and steel] the modulus [of elasticity] is practically constant * * ." Again, Kent§ quotes the late J. B. Johnson, M. Am. Soc. C. E., as follows: "The modulus of elasticity is the most constant * * * property of all engineering materials." The treatment of the subject is too brief, and is generally inadequate. Any one having accepted such ideas receives a shock when he turns to Kirkaldy's "Report L", or the reports on "Tests of Metals" at Watertown, and notes that the sets for iron and steel eye-bars run roughly from 2 to 17% of the elongation under the stresses applied before the elastic limit is reached. Fig. 7 illustrates the average percentage of sets on first loading of a few specimens of steel and wrought iron.

It may be that this dereliction of the elastic properties can be eliminated by a single loading to a slightly greater stress than that at which the sets were measured. Some tests|| which the writer has had to look after seem to indicate that several repetitions of stress are required; but, for steel, it is certain that the elastic properties are rendered nearly perfect by application of stress, and that this can be carried up to and beyond the elastic limit or yield point exhibited

* One of the most difficult features of the visco-elastic properties of steel is exhibited by the sudden tensile test which causes greater elongation than when similar material is pulled slowly. Indeed, Trouton and Rankine (*Philosophical Magazine*, Vol. 224, 1904, p. 536, etc.), speaking of their experiments on viscous traction, conclude that "the explanation of the phenomena must be sought for on altogether different lines from those based on mixed viscous and elastic action in the way commonly attempted."

† "American Handbook for Electrical Engineers," p. 1418.

‡ "Mechanical Engineers' Pocket-Book," p. 351.

§ *Ibid*, p. 352.

|| Not published.

Mr. McCulloch.

on primary loading. Under certain conditions, this perfecting of the elastic properties by application of high stress may enhance the utilitarian available strength remarkably.

Returning to the experiments that showed the necessity for more than one application of stress, it may be said that the measures were commercial testing, and had to be made with dispatch, so that what is known as "Webber-effect" or *Nachwirkung* may have complicated the observations. Indeed, some few slowly conducted tests clearly indicated this "afterworking" of elastic recovery.

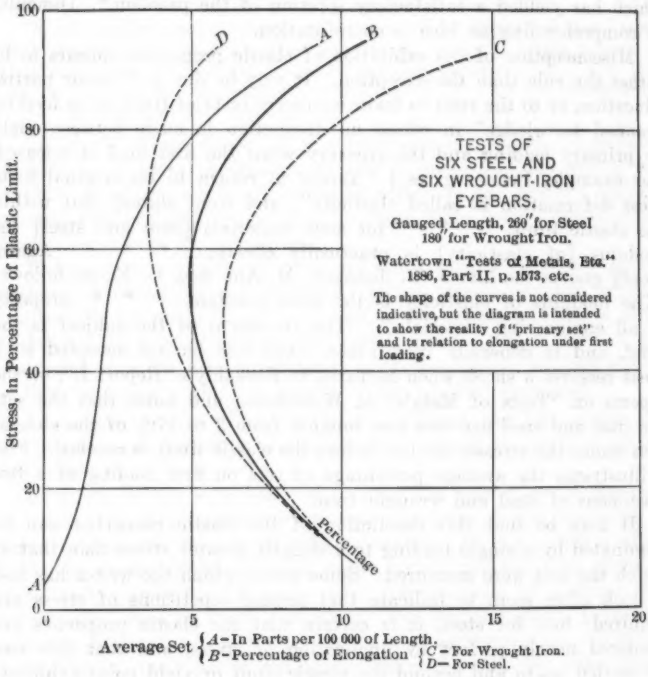


FIG. 7.

It would be wrong to dwell on this set on primary or first loading were it not for the fact that it is a common mistake, and ignorant men entrusted with the framing of specifications for steel frequently err by inserting a requirement that on removal of the first load no evidence thereof shall remain in the form of permanent set. In his early experience the writer was told to draw up a specification for coiled (helical) steel springs. It was promptly rejected by all makers

because of a requirement that there should be no set on first loading. For more than twenty years this erroneous idea of "no set on first application of load" has been specified, and the writer has become convinced that the proper and true idea of "primary set" has not been given the credence that is warranted by the ready accessibility of its proof. This type of set is non-indicative of quality. Philip Schidrowity, writing on "Rubber", says that even there it is not indicative, but that the rate of decrease in the area of the hysteresis loop on successive loadings does connote quality. Dr. Steinmetz has called attention to this "physical hysteresis" and its analogy to the electro-magnetic phenomenon of the same name.* Mr. McCulloch.

Under nearly every condition of the application of steel to its service, this set on first loading ("primary set") is wholly negligible. However, in the case of columns, the writer would attribute much of their notable dereliction on first loading to this primary set in conjunction with the variability of the elastic modulus where that property is measured by either yield or recovery from step to step of first loading, as shown by many of the Watertown tests.

One often hears expression of the erroneous idea that straining steel is injurious, but stress which does not over-strain puts steel in "a state of ease" where it acts as though it were nearly perfectly elastic. Even over-straining steel (if not carried to a ruinous extent), when allowed a reasonable time to recuperate before the next application of high (but lower) stress, will exhibit the characteristics of the "state of ease". The set will not be increased to more than that produced by the earlier strain; that is, repetitive loadings do not cause additional or cumulative distortion in steel. In this, steel is wholly unlike some of the soft metals, and, therefore, is vastly superior to them as a structural material.

The notable and great enhancement of the physical properties of steel by strain are well known, although many attempts have been made to ascribe the benefit to some more recondite phenomenon. Forging, rolling, drawing, etc., all improve steel, if the pressure is applied in a manner that causes a general flow; that is, when the pressure is not applied in a manner that causes high local stress.† When high local shears are caused, the general quality may be injured, but, by skillful application of this principle, Herr Mannesmann

* *Engineering News*, December 5th, 1912, p. 1051.

† Caution is required. The homogeneity of steel is not "perfect", but the imperfections (some visible under the microscope) are so small when compared with those of wrought iron, that there is some propriety in the use of the term "homogeneity". The imperfections of the two materials are so dissimilar that they belong to different orders of magnitude (to use mathematical language). Even the slag enclosures of wrought iron do not preclude benefit from stress. Dr. A. Leon, writing on "Irregular Strains Due to Non-Homogeneity of Materials" (Int. Assoc. for Testing Materials, No. 9, VIII, 1909), indicates that, although the material adjacent to cavities and enclosures may be at stresses vastly higher than the average applied stress, this localizing of stress largely disappears when ductile metals are strained beyond the yield point.

Mr. McCulloch. turned the failure of his planishing mill into the notable advance that bears his name as a method of producing hollow blooms which are of very high quality. Wire which has been drawn has been subjected to great traction at the same time that great external pressure was applied by the die. The stresses are far above the yield point, and the quality of the material is enhanced to a measure greater than the elimination that annealing involves when putting in shape for suspension bridges. Although what is spoken of as heat treatment is doubtless mostly due to rapid cooling through the critical range, it has long been the writer's idea that the rapid cooling of the surface induces a great pressure on the interior, thus producing a benefit from stress, perhaps analogous to, but less profound than, fluid compression.

This fact of paramount importance remains, that steel is improved by strain. When the stress exceeds the yield point and is released for a sufficient time, the enhancement of the quality of the steel is immediately available and of great practical value, regardless of whether the stress is applied by the maker or user. High straining, if done in an ignorant fashion, may injure steel or other materials.

Mr. Jonson. ERNST F. JONSON,* ASSOC. M. AM. SOC. C. E.—There are two points in this paper on which the speaker wishes to comment.

The author seems to conceive of the flow which takes place in ductile metals when stressed beyond the yield point as if it were due to a change in the metal from the solid to the liquid state. The energy required to bring about this change of state he seems to regard as a part of the work done by the load. This, however, is not in accordance with the facts. All the work done by the load previous to the beginning of flow is stored in the test piece as elastic energy, and, therefore, is not utilized in changing the state of the metal, as is the heat which is transformed into fluidity energy when a solid melts. One of the main distinctions between a solid and a liquid is the difference in their elastic limits. The elastic limit of the solid has a certain positive value, but that of a liquid is zero. As the temperature rises, the elastic limit diminishes until it becomes zero at the melting point. Flow takes place in solids, as well as in liquids, only when the shear exceeds the elastic limit; liquids flow under an infinitesimal shear because the elastic limit is zero; solids flow only when the shear is infinitesimally in excess of the elastic limit of the material.

The author's use of the words "sliding" and "slipping" in connection with flow seems to be unfortunate, because flow is something entirely different from slipping. Flow consists of an indefinitely extended, angular deformation, which is the resultant of a diagonal tension and a diagonal compression perpendicular to it. As soon as slipping takes place the specimen breaks and the flow stops. The fundamental differ-

* New York City.

ence between slipping and flow is best realized by remembering that the former is resisted by friction, and that the resistance produced by the latter is viscosity, two entirely different phenomena, following different laws; friction is a function of the pressure, and viscosity is independent of pressure and is a function of velocity and temperature. Mr. Jonson.

Metals are not uniform in elastic limit in all their parts, and therefore they do not flow uniformly. The flow naturally begins in the particles of the lowest elastic limit; but, owing to the fact that flow raises the elastic limit, the flow soon spreads to the particles of higher elastic limit. As the flow thus spreads throughout the mass of the metal, its quantity increases, and therefore the product of the elastic limit and the cross-sectional area increases at a decreasing rate until it ceases to increase and begins to decrease. The point at which this change takes place is the ultimate strength. When the elastic limit becomes equal to the cohesion of the material, no further stretching is possible and rupture occurs.

The author seems to be under the impression that shear is a peculiar kind of stress, an elemental stress, but shear is rather the resultant of two or three linear stresses acting at right angles to each other. A simple or two-dimension shear arises when two principal stresses of opposite kind but of the same magnitude act in the same point. A compound or three-dimension shear arises when two principal stresses of the same kind and magnitude, and a third principal stress of the opposite kind and twice the magnitude or greater, act in one point. Flow begins when the shear reaches the elastic limit, but rupture is not produced by shear. Tensile stress alone produces rupture, for rupture cannot be produced by compressive stress. The only condition under which pure compressive stress is possible is that of a specimen immersed in a liquid under hydrostatic pressure, and experiments have shown that rupture is not produced in this way. Hence, it follows that rupture is due to the tensile components. Since tensile or compressive stress may exist without shear, but no shear can exist without both tensile and compressive stress, it is evident that shear is not a simple form of stress.

T. E. Stanton,* Esq. (by letter).—The writer has read this paper with great interest, and considers it a valuable contribution to the literature on the strength of materials. There are one or two statements, however, to which he would like to call attention because he thinks that recent researches have shown that they need some modification. On page 1431, it is stated that in pieces subjected to alternate direct tension and compression, an increase in the rapidity of the alternations reduces the safe range of stress. Again, on page 1456, referring Mr. Stanton.

* Teddington, Middlesex, England.

Mr. Stanton. to the National Physical Laboratory tests on alternations of direct stress (Table 4), the author states:

"In structural members, however, as the frequency of the alternations is much less than in these tests, and as periods of rest are interspersed between alternations, it is fair to assume that the limits between which the stresses can alternate are greater than in the tests."

These conclusions are apparently based on Reynolds and Smith's experiments,* in which a very large reduction in the safe range of stress was observed as the rate of alternation increases from 1 400 to 2 500 per min. The writer believes that it is now coming to be recognized that these results were peculiar to the testing machine used. At any rate, in the modified Wöhler method of test, which has been used extensively at the National Physical Laboratory during the last 10 years, no trace of a speed effect on the safe range of stress has been found below 2 and 2 200 alternations per minute. Further, periods of rest between series of alternations do not appear to have an appreciable effect on the total number of alternations for fracture. Again, in discussing the question of fatigue, on page 1433, Mr. Prichard states:

"* * * the greatest strength of steel or wrought iron under repeated stresses is beyond, and in some cases far beyond, the yield point."

Assuming that by the term "repeated stresses" is meant stresses varying from zero to an upper limit, the statement needs modification, as in many steels the maximum safe stress for repeated loadings from zero is well below the yield point. For example, in Mr. Bairstow's valuable paper† on the range of the elastic limits, with which Mr. Prichard does not seem to be familiar, experiments on an axle steel with a yield point of 24.9 tons per sq. in. showed that in repeated loadings from 0 to 23.2 tons per sq. in. no sign of want of elasticity occurred until after 6 000 loadings, when a permanent extension of the order of the yield took place.

The writer thinks it is not sufficiently recognized by engineers that the phenomenon of repeated loadings of a structure from zero to an upper limit is only a particular case of cyclical variations of stress in which the lower limit of stress is zero, and that it has been proved conclusively that the safe range of stress under such conditions is appreciably less than in the case of reversals of stress between equal values of tension and compression.

Mr. Howe. HENRY M. HOWE,‡ ESQ. (by letter).—The Society is to be congratulated on this very clear and interesting exposition of the current theory of plastic deformation, and on the reasonableness of the inferences drawn from our present very fragmentary knowledge. This

* *Phil. Trans.*, Royal Society, Vol. 199, p. 265.

† *Phil. Trans.*, Royal Society, Vol. 210, p. 35.

‡ Bedford Hills, N. Y.

Mr. Howe.

theory, of course, lacks full confirmation, failing, for instance, to explain why, after a heating so gentle as to intensify the hardness caused by the deformation, no traces of the amorphous metal along the slip planes can be detected, though, according to this theory, it ought to be prominent.* In particular, one welcomes the assertion on page 1459 that the real factor of safety in such cases must be based on the elastic limit and not on the tensile strength. This, of course, refers to what one may call "hypo-elastic" uses, as in structures which, in the nature of the case, must not be subjected to stresses great enough to cause important plastic deformation. That this is true of bridges and buildings is evident. For what may be called "hyper-elastic" uses, the tensile strength may possibly be a more useful measure of fitness. A hoisting cable, for instance, need not be unfitted for its use by a degree of permanent stretch which will raise its elastic limit very greatly.

Perhaps our view of the whole subject may be clarified by looking at the hyper-elastic part of a tensile stress strain diagram as approximately the locus of the elastic limit. If, at any moment after the yield point has been passed, the stress is released and re-applied, no new permanent set arises until the new stress reaches approximately the maximum hitherto applied, which means that the stress at that point in the diagram is approximately the existing elastic limit of the test piece. The first sentence of this present paragraph is only a generalization from this particular case. In this view, the tensile strength is the maximum elastic limit which the material is capable of developing under other roughly similar conditions, as when a cable is slowly strained beyond its primitive elastic limit up to the point at which it finally breaks.

In this view, the drop of the diagram at the yield point and its later rise represent the algebraic sum of (*A*) the weakening which the test piece is undergoing, (1) because of its progressive loss of section, and (2) from the mobilizing of part of the metal, initially all crystalline; and (*B*) the strengthening which each unit of section is given by the solidification of the momentarily mobilized metal, or, in short, by the plastic deformation itself. At the beginning of the yield, the curve may drop sharply because now the section is decreasing rapidly and mobilization is rapid, whereas the strengthening, which will soon follow this mobilization, has not yet reached considerable proportions. The flat part or jog means that here the strengthening of the material by the deformation is just equalling the loss of strength represented by the simultaneous mobilization of crystalline metal plus the loss of section. This part is probably never truly flat, but a series of waves. The rise of the curve, like the up side of each of these waves, means that now the strengthening caused by the deformation is ex-

* "Metallography of Steel and Cast Iron", pp. 526, 625, 626, 634, and 682.

Mr. Howe.

ceeding the weakening due to the loss of section, and the final drop on passing the so-called tensile strength or maximum load, like the down side of each of the waves in the jog, means the reverse. In this view the rise of the diagram beyond the jog changes from a miracle to an almost prophesiable phenomenon. On this view, why should hypo-elastic service stresses be based on the so-called tensile strength, which is only a certain strength capable of being developed under conditions wholly different from those of service, and wholly unapproachable in service, instead of on the elastic limit, which is the only strength existing in that service?

The writer had hoped that Mr. Prichard, in addition to showing the practical ineffectiveness, for hypo-elastic services, of the elevation of the elastic limit, or, more briefly, the strengthening, by service deformation itself, would have touched on the important question of the effectiveness of such elevation by pro-service deformation, such as cold-rolling and cold-drawing. One of the important questions before us is as to the limits within which the strengthening caused by pro-service deformation is effective. There is much to suggest that, under indefinitely repeated reversals of stress-sign, any such pro-service strengthening is wholly ineffective, and that it is the primitive elastic limit alone that determines the endurance, that is, the fatigue strength. Indeed, there are well-informed and just writers who imply that this ineffectiveness is established and certain. If this is true, then the measure of strength of axles, shafting, etc., is given, not by the elastic limit as observed in the material as delivered, but by that determined after the removal of all pro-service elevation of the elastic limit by plastic deformation, and hence the reception tests should be made on pieces annealed say at 600° cent. for the purpose of removing the strengthening caused by cold- or cool-rolling, or any other form of plastic deformation. The question whether this pro-service strengthening is effective or fictitious under indefinite reversals of stress-sign is of an importance which calls for decisive tests. If ineffective under indefinitely repeated reversals, then the question comes up, for bridge engineers and the like: "What are the limits within which this pro-service strengthening is effective, and beyond which it ought to be effaced before making the reception tests, by 600° annealing or its equivalent?"

There are a few minor points which may merit attention. Two reasons suggest themselves for considering the use of the term "proportional limit" rather than "elastic limit" for the limit determined directly from the stress-sign diagram. First, "elastic limit" has been used so widely and persistently in the sense of "yield point" that its use may still leave the reader in doubt as to which is meant, whereas there is no such doubt about "proportional limit". Second, that which we know to be the elastic limit is the maximum stress which just fails

to cause a permanent set. It is true that every refinement of measurement lowers the elastic limit which we observe by releasing the load and remeasuring the measured length. Hence the ideal elastic limit is that observed with perfect instruments, and hence it can never be actually determined, but only approximated. Nevertheless, its determination is essentially different from that of the proportional limit, which, for like reasons, can only be approximated. If it is true that the proportional limit and the elastic limit coincide—and, however probable this may be, it is not yet known to be a universal law—then it is a purely empirical observation. Might it not be well to use these terms "elastic limit" and "proportional limit" each as indicating the way in which the determination is made? There are those who actually determine the elastic limit by release and remeasurement, and prefer this procedure. Is it not a little arbitrary to deprive the term "elastic limit" of its natural and obvious application to such determinations, and apply it by force to the proportional limit? What is the gain to offset these disadvantages? Mr. Howe.

Even if rupture through fatigue is always preceded by tearing, does it follow, as suggested on page 1432, that fatigue is always accompanied by tears? Why may not a very important degree of true fatigue occur before even sub-microscopic tears form?

What is gained by complicating the notion of slip by introducing rotation, as on page 1437? The conception of Ewing and Rosenhain, as well as that of Beilby and other writers, if the writer understands them aright, is that slip itself is purely vectorial, consisting of translation absolutely without rotation, though it may be accompanied by rotation which in itself leads to a radically distinct mechanism of deformation, *viz.*, twinning.

Later results have thrown very serious doubt on the inference drawn from Howard's experiment (page 1445) that hydraulic pressure does not affect the properties of metals.

Is the conception (page 1450) that during continued flow there is a prolongation of the mobile state of certain individual particles, as easy and probable as the alternative conception that, during such flow, each particle passes with relative rapidity from the crystalline through the mobile to the rigid amorphous state, and that the continuousness of the flow represents the succession of various sets of particles in this progress?

On either view flow is replaced by rupture as soon as the supply of residual crystalline metal is so far depleted that it no longer suffices to generate, under continuing deformation, mobile metal fast enough to heal the cracks which that same deformation is causing in the metal which has passed beyond the mobile into the rigid amorphous state. The conception that the mobile metal is prevented by continued deformation from setting would imply that such continuance would

Mr. Howe. defer greatly this time of depletion, or, in other words, increase the ductility very greatly, which is not true.

Mr. Irwin. A. C. IRWIN,* ESQ. (by letter).—The author is to be congratulated on having produced a paper of exceptional merit. It should be of great interest to engineers as an enlightened exposition of a convenient theory and a clear statement of certain changes in the properties of wrought iron and structural steel produced by strain. It is hoped that it will stimulate engineers to a more active interest in the influence of the proximate constituents on the physical properties of iron and steel.

It is hoped also that the use of formulas for fatigue in the design of ordinary static structures will be finally discarded, as the theory advanced in this paper, as well as other theories well authenticated by experiments, show that in ordinary static structures, where no reversal of stress occurs, "fatigue" either does not exist, or is of no consequence.

The theory proposed by Mr. Prichard is open to some logical objections, as well as being apparently at variance with certain knowledge obtained and well-proven by various investigators. According to this theory, the internal structure of iron and steel is made up of "grains", "particles", or "crystals"—using these terms interchangeably—and an intergranular material which exists as a film on the surface of each grain, and fills the intergranular spaces. The "grains", "particles", or "crystals" possess and retain the property of almost perfect elasticity under either elastic or non-elastic deformation of the piece. The film, or intergranular material, however, becomes a viscous liquid when the piece is strained beyond the elastic limit, but re-solidifies with great rapidity on the release of the straining forces. The groundwork of Mr. Prichard's theory may be found in the first paragraphs on pages 1430 and 1432, and the second paragraph of page 1450, from which the following conclusions may be drawn:

First.—Elastic deformation is effected wholly by deformation of the "grains", "particles", or "crystals".

Second.—Non-elastic deformation or permanent set is effected wholly through plastic flow of the intergranular material, which causes the grains to slide over each other into new positions—the grains themselves undergoing only elastic deformation and regaining in time their original dimensions, shapes, and bulks, on the cessation of the external straining forces.

Third.—Failure of an over-strained piece takes place wholly by failure of the intergranular material, the grains themselves retaining their integrity and their nearly perfect elasticity.

Fourth.—The recovery of a piece from over-strain is dependent on the extent of elastic deformation of the grains. The push or pull

* Chicago, Ill.

exerted by the forces of elastic return of the elastically strained grains on the "viscous liquid" is all that prevents its immediate re-solidification on the release of the external straining forces, and the magnitude of these forces of elastic return is sufficient to prevent complete re-solidification for a considerable time. Mr. Irwin.

It seems that, if the last conclusion is true, the following should also be true, *viz.*, that elastic change of form of the grains of a piece, under the action of external straining forces, exerts a push or pull on the intergranular material of such magnitude as alone to cause a degree of liquefaction; and that, as the grains suffer considerable elastic deformation at stresses near to, but below, the elastic limit of the piece, a certain amount of liquefaction of the intergranular material must occur, even in elastic strain, to accommodate the changed dimensions and forms of the grains. Some period of time would then be required for recovery from elastic strain, just as in non-elastic strain, because, in both cases, there is the retarding effect of the forces of elastic return of the elastically strained grains; and, furthermore, it would be expected that the time for recovery from elastic strain would be some considerable portion of the time for recovery from non-elastic strain, the difference in the two cases being merely one of degree in the magnitude of the same kind of strain. The time required for recovery from any considerable over-strain may be hours or days, and we would be led to expect that the time for recovery from elastic strains produced by stresses at nearly the elastic limit would be at least quite appreciable. On the contrary, no appreciable time is required for recovery from elastic strain, and, therefore, we conclude that no liquefaction or flow of intergranular metal takes place. Further, as liquefaction and retardation of solidification are but counterparts of each other effected by exactly the same kind of forces, *viz.*, those of elastic strain and return, and as no time is required for solidification because of these forces under elastic strain, we conclude that no such liquefaction as that described by the author takes place under non-elastic strain.

The second and third conclusions derived from Mr. Prichard's paper, *viz.*, that non-elastic deformation and final failure are respectively dependent wholly on the plastic flow and failure of the intergranular material, and, accordingly, that the grains themselves retain their integrity and nearly perfect elasticity, are at variance with views—held by metallographers—which have been well proven by experiments.

"Under a progressively augmented strain, rupture takes place, not at the crystal boundaries, but through the crystals themselves."*

* "The Crystallization of Iron and Steel", by J. W. Mellor, p. 91.

Mr. Irwin.

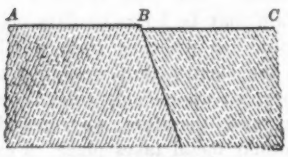
"Fractures in fact are always trans-crystalline in properly treated steel, an intergranular fracture being clearly pathological, as for instance, in burnt steel."*

"The plastic deformation of all kinds of iron is primarily in their ferrite. The cementite present, whether pearlitic or proeutectoid, may break or bend, but it probably does not yield plastically to any large degree."†

"The manner in which a metal yields when the strain exceeds the elastic limit is by slips which occur in the cleavage or 'gliding' planes of the individual crystals."‡

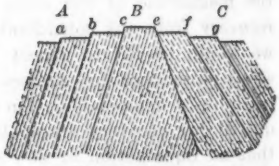
The micro-structure of wrought iron and steel is essentially crystalline. In wrought iron and low-carbon steel, this crystalline structure is most perfect in the free ferrite crystals which exist in predominant quantity. Free ferrite crystals are true crystals, in that they are composed of an enormous number of crystalline elements, molecules, or "brickbats", arranged in perfectly regular tactical order.

These elementary molecules or "brickbats" composing a crystal grain have their like axes and faces all parallel in any given crystal. The axes of the "molecular brickbats" in adjoining crystals, however, are not usually parallel, so that each crystal has its own individual orientation.§



(Before Plastic Strain)

FIG. 8.



(After Plastic Strain)

FIG. 9.

The intra-crystalline slips which are almost wholly responsible for the plastic deformation of wrought iron, low-, and medium-carbon steel, as described and proven by Rosenhain, Ewing, and others, are illustrated by Figs. 8 and 9.

A-B and *B-C*, Fig. 8, represent the polished surfaces of two adjoining ferrite crystals before plastic strain of the piece. The dotted lines represent glide planes of the crystal, that is, the dividing planes

* "The Metallography of Steel and Cast Iron", by Henry M. Howe, 1916 ed., p. 293.

† "The Metallography of Steel and Cast Iron", by Henry M. Howe, 1916 ed.

‡ Encyclopædia Britannica, 11th ed., Vol. XXV, p. 1018; see, also, "The Crystalline Structure of Metals", by Messrs. Ewing and Rosenhain, *Phil. Trans.* of the Royal Soc., 1900; "The Microscopic Analysis of Metals", by F. Osmond, 1913; W. Rosenhain, *Journal*, Iron and Steel Inst., 1914, No. II, p. 367; and "Introduction to Physical Metallurgy", by W. Rosenhain, 1915.

§ A popular and lucid explanation of crystallization in a pure metal and the effects of straining Swedish iron, together with an electrical theory of plastic strain and fatigue, is given by James Alfred Ewing in *Memoirs and Proceedings*, Manchester Literary and Philosophical Soc., Vol. 51, 1906.

between groups of tactically arranged molecular "brickbats". In Fig. 9, at *a*, *b*, *c*, *e*, *f*, and *g*, are steps in the polished surfaces of the crystals after plastic strain of the piece. These steps are produced by the slip of the molecules on one side of a glide plane over those on the other side of it. The dark lines shown by these breaks in the polished surface of each crystal, under the microscope, have been called "slip bands" and are the "slips" to which the author refers on page 1437. As previously noted, these "slips" are intra-crystalline and not inter-crystalline, as we are led to believe from the paper. Mr. Irwin.

Of so-called impurities that always occur in commercial iron, *viz.*, carbon, silicon, phosphorus, manganese, and sulphur, silicon and phosphorus are retained by the iron in solid solution, and carbon, manganese, and sulphur form compounds with the iron (ferrite) and with themselves, which compounds are rejected to the crystal boundaries or along crystallographic planes while the metal is passing through the critical range; that is, during conversion to the final crystalline structure of ordinary temperatures.

The rejected impurities are in the form of definite compounds with iron; the carbon as carbide of iron (Fe_3C, or cementite), the manganese as manganese sulphide (Mn S) and manganese carbide (Mn_3C). The cementite combines with ferrite to form pearlite, which appears as a lamellar structure of very thin alternating plates of cementite and ferrite. The ferrite not required to form pearlite is known as "free ferrite", and crystallizes generally in the cubic system.

Steel having less than about 0.85% of carbon is known as hypo-eutectoid, and is composed essentially of pearlite and free ferrite. These constituents vary proportionally with the carbon present. Thus, pure iron is made up of ferrite crystals with no pearlite present. With a very slight carbon content, the ferrite crystals will have interspersed between them small pearlite masses. With increase in carbon, the pearlite increases in proportion until a connected net is formed, the meshes of which are completely filled up with ferrite crystals. With further increase in carbon, the pearlite net occupies more and more space until it becomes the meshes of a ferrite net, and finally the steel is composed entirely of pearlite. Steel composed entirely of pearlite is known as eutectoid steel, and results when the carbon content is just sufficient to form cementite in such quantity as to require just the total ferrite present to form pearlite. The carbon content of eutectoid steel is variously given as from about 0.82 to 1 per cent. If the cementite is in excess of the quantity required to form pearlite, the resulting steel is known as hyper-eutectoid. In the latter case excess cementite is known as "free cementite".

The physical properties of ferrite, pearlite and cementite, according to Sauveur, are as given in Table 6.

Mr. Irwin.

TABLE 6.—PHYSICAL PROPERTIES OF FERRITE, PEARLITE, AND CEMENTITE.

Constituents.	Tensile strength, in pounds per square inch.	Elongation, percentage in 2 in.	Hardness.
Ferrite	50 000	40	Soft.
Pearlite	125 000	10	Hard.
Cementite	5 000 (?) *	0+	Very hard.

* It seems very probable that cementite has a considerably higher tensile strength than 5 000 lb. per sq. in., and an appreciable elongation percentage.

In Table 7 is reproduced the author's Table 1, except that computed percentages of ferrite, pearlite, and cementite are substituted for the columns for yield point. It will be noted in this table that the elastic limit increases, in general, with the percentage of pearlite, and indicates clearly the influence of this constituent on the elasticity of the material. It is well known, too, that the total percentage of elongation at ultimate decreases, in general, with increase in the carbon content, that is, with the increase in the percentage of pearlite. This is to be expected from the comparatively low elongation of pearlite given in Table 6. Moreover, the percentage elongation given for pearlite is much less than that obtained from low-carbon or "medium" steels. It is evident, therefore, that, although the intergranular material, that is, pearlite, greatly influences the elastic strength, it is not responsible for the ultimate elongation of such steels, and that we must credit the ferrite with by far the most important role in plastic deformation.

TABLE 7.—TESTS MADE AT WATERTOWN ARSENAL, AND GIVEN IN THE REPORTS FOR THE YEAR 1886, PP. 1635-1653, AND FOR 1887, PP. 802-822.

Marks.	Composition, Percentage of:			Modulus of Elasticity.		Elastic Limit.		Constituents.	
	C.	Mn.	Si.	Compression, in pounds per square inch.	Tension, in pounds per square inch.	Compression, in pounds per square inch.	Tension, in pounds per square inch.	Percentage of free ferrite.	Percentage of pearlite.
833	0.09	0.11	....	30 120 000	30 151 000	30 500	30 000	89.2	10.8
123	0.20	0.45	....	30 303 000	30 151 000	37 000	39 500	76.0	24.0
782	0.31	0.57	....	30 612 000	30 000 000	44 500	46 500	62.8	37.2
795	0.37	0.70	....	31 250 000	30 151 000	47 000	50 000	55.6	44.4
805	0.51	0.58	0.02	30 075 000	30 000 000	57 000	58 000	38.8	61.2
797	0.57	0.93	0.07	30 201 000	30 104 000	55 500	55 000	31.6	68.4
823	0.71	0.58	0.08	31 034 000	30 088 000	55 500	57 000	14.8	85.2
750	0.81	0.56	0.17	30 000 000	29 923 000	74 500	70 000	2.8	97.2
756	0.89	0.57	0.19	30 612 000	29 864 000	76 500	75 000	1.0*	99.0
334	0.97	0.80	0.28	30 822 000	29 817 000	83 000	79 000	2.7*	97.3

* Free cementite.

When wrought iron, or low-carbon or "medium" steel, is strained within its elastic limit, the pearlite takes stress in proportion to its strength to resist elastic deformation, and lends aid to the weaker ferrite crystals. So long as the elastic limit of the pearlite is not passed, there is no slip in the ferrite grains, excepting, perhaps, in isolated grains, the orientation and disposition of which, with respect to the surrounding pearlite and adjacent ferrite grains, presents them in the most unfavorable position to resist the straining forces. When the elastic limit is passed, small slips or cracks occur between or through the plates of the lamellar pearlite, or at the boundaries between ferrite and pearlite, or at any of these places in combination, so that a greater proportion of stress is thrown on the ferrite. Moreover, the support accorded to the ferrite by the pearlite is thus removed, to a degree, and the result is slip along the crystallographic planes of the ferrite crystals. This slip is accomplished by the disarrangement of molecules along the glide planes affected—the greater the slip the greater the disarrangement and the more molecules affected. Disarranged molecules appear as an amorphous material in the planes of slip, and, under severe over-strain or many reversals, at the intersection of the slip planes with the boundaries of the crystal grains. This amorphous material forms a very thin film—of perhaps molecular thinness—along the slip planes, and when extruded at the grain boundaries may fill cracks in the intergranular material that register with the planes of slip. If the over-strain is slight, the disarranged molecules will rearrange themselves, in something like their former orientation, with those of the crystals from which they were broken; but, in the case of considerable over-strain, their disarrangement is so great that they remain in an amorphous condition. This amorphous metal is hard and quite strong, and binds the parts of the crystal so strongly together at the planes where slip occurred that the planes on which no slip has occurred are planes of weakness in comparison. Thus, the weak portions of the piece have their strength increased and a greater application of stress will be required to produce further plastic deformation than was required in the first instance—the deformation being now effected by slip along planes not thus far affected. This process results in the development or increase in the elastic strength of the piece as a whole. Mr. Irwin.

Under severe over-strain, slips occur, not only along the three principal glide planes of the crystal, but along other planes, so that the polished surface of the crystal becomes checkered with numerous intersecting slip lines. Final fracture of the piece is initiated by the opening up of a minute crack along a slip plane of some crystal which registers with a minute crack or slip in the pearlite. The crack quickly widens and propagates to other crystals through the more or less broken up pearlite, until fracture takes place. If the

Mr. Irwin.

stresses producing final failure are applied quickly, as by shock or by continuous rapid reversals, "the resulting fracture exhibits the crystal faces upon which slip has taken place as a number of bright facets resembling those produced in a 'brittle' shock fracture, and it is this appearance which has led to the mistaken idea that alternative stresses cause metal to become crystalline."*

The time of recovery from over-strain will depend on the degree of disarrangement of the molecules and the hindrance to their adoption of the most stable arrangement possible under their condition of confinement.

The process of failure by "fatigue" under repeated reversals of stress is explained on the same basis of slip in the crystalline grains. The amount of slip, and hence formation of amorphous metal, is greatly augmented by reversal of the direction of slip before the disarranged molecules can adapt themselves to the new conditions, that is, before the amorphous metal can harden. The slips in the weak crystal are thus continued until a crack is developed which spreads to other crystals. Fracture finally occurs at a unit stress for the piece much below that required for a stress in one direction intermittently applied. In the latter case, the primitive elastic limit arising from slight slips in unfavorably oriented and located crystals is eliminated, and the normal elastic limit developed in the piece. Therefore, the use of fatigue formulas in proportioning the members of static structures under uni-directional stress has no rational basis. Moreover, their use produces a poorly balanced design. If fatigue formulas are recommended for the reason that they allow a margin for impact, the reply is that such use of them relegates to the realm of unscientific guessing a matter about which fairly definite knowledge is available in the various reports of the Sub-Committee on Impact and Secondary Stresses of the American Railway Engineering Association.

In addition to making the provision for dead load relatively too small for long-span bridges, "fatigue" formulas (and others) result in making the provision for live load in certain members relatively too small in short- and medium-span bridges. This becomes apparent when it is desired to load the bridges to their maximum carrying capacity.

In general, the writer believes that a properly balanced proportioning of bridge members is most likely to result from separate consideration of all stress-producing factors and the use of the highest unit stress to which the material may safely be subjected. The assumed loading should then be the heaviest that will ever come on the structure, and the total stress that obtained by taking the sum of dead load, live load, live-load impact, and the secondary stress

* "An Introduction to the Study of Physical Metallurgy", by W. Rosenhain, p. 254.

resulting from the deformation produced by the primary stresses. With this method of design, it will not be necessary to replace bridges because of certain weak members before they are loaded to the maximum carrying capacity of the majority of their parts. Mr. Irwin.

DAVID A. MOLITOR,* M. AM. SOC. C. E. (by letter).—The author, in this paper, brings to the attention of the Profession a valuable collection of data which should serve to strengthen similar arguments and opinions previously advanced by others. Mr. Molitor.

It should also materially assist in relegating to the past, several formulas which have retained a place in specifications for no better reason than a lack of moral courage to cast them out.

One very important matter, also clearly indicated by column tests other than those given in the paper, is the remarkably close agreement between the ultimate strength and elastic limit, or better, limit of proportionality. Of course, this can only be true for columns of considerable length; for very short columns or cubes, the ultimate strength is practically the same in both tension and compression and, for columns still shorter than the side of a cube, the ultimate compression may be called the flowing strength, and is the highest strength attainable. These three classes of failure under compressive stress are explained by combined bending and direct stress for long columns, true direct stress for cubes, and flowing stress for plates.

In tension specimens we observe an elastic limit approximately the same as for compressive stress, though the ultimate strength is about twice as great.

We are naturally forced to the admission that any stress in excess of the elastic limit, whether it is tensile or compressive, will result in some permanent injury to a structure, owing to a permanent deformation due to the excess stress. This may not be true of a piece not forming part of any structure, but we are usually concerned with structures consisting of a multiplicity of pieces in which each piece has a certain duty to perform. Hence, a sufficiently great number of repetitions of stress in excess of the elastic limit must ultimately cause failure, a fact clearly demonstrated by the experiments of Wöhler, Bauschinger, and others.

Therefore, in so far as we are concerned with structures, the elastic limit of the material, both in tension and compression, must also be the limit of permanent usefulness of any given structure, even though the ultimate strength in tension for a single condition of stress, may be double that in compression for any of the component parts.

In the case of the collapse of any bridge, as a result of gradually applied overload to the point of failure, the initial failure will always be in the highest stressed column and not in a tension member, pro-

* Detroit, Mich.

Mr. Molitor.

vided the design was based on a uniform factor of safety with respect to the elastic limit of the material used. To avoid this, columns would have to be made almost twice as strong as common practice demands. The Quebec Bridge failure clearly points to this conclusion.

However, a structure designed for unit stresses approaching the elastic limit simultaneously in all its members for a 100% overload, would obviously have a factor of safety of 2 throughout, would be uniformly safe, and would be considered a prudent design. Unfortunately, most bridges have not been designed in this way in the past, and generally have some members in which the unit stress resulting from overload increases at a greater rate than the rate of increase in the load applied. All members, in which the dead-load stress is of opposite sign to that of the maximum live-load stress, belong to this class; likewise all members, the live-load stresses of which are produced by a partial loading of the span.

Various specifications have been written to obviate this difficulty, but, in most cases, these involve the addition of metal to certain members, without any logical system.

The writer has devised a method of design which effectually overcomes the difficulties here mentioned, with the idea of producing structures of uniform strength throughout, for an overload of 100% in the live load, without ever exceeding the elastic limit of the material. As a basis of design, the allowable unit stress in tension is taken as $f = 16\,000$ lb. per sq. in., and, in compression, $f = 16\,000 - 60\,\frac{l}{r}$ for fixed ends, and $f = 16\,000 - 80\,\frac{l}{r}$ for one or two hinged ends. This method is described here for the first time.

The stresses in the chord members and end posts are directly proportional to the total loads covering the entire span length, hence to double the live load will increase the unit stresses in these members by a constant percentage, always less than 100% so long as the dead-load stress is greater than zero. However, this same overload will generally increase the unit stresses in the web members in a much greater ratio than for the chords, hence the following specifications:

Members subjected to dead- and live-load stresses of the same character, and in which the maximum live-load stress is produced by the maximum live load covering the whole span, shall be proportioned for the sum of the dead-load, live-load and impact stresses, on the basis of the allowable unit stresses, f, given for tension or compression, as the case may require. This applies generally to chords and end posts. With the correct section thus found and a total stress, $s' =$ (dead load) $+ 2$ (live load) $(1 + I)$, find the unit stress, f', resulting from 100% overload in the live load; and also find the factor, $k = \frac{f'}{f}$ (always less

than 2), which represents the ratio of increase in the unit stress, f, due to 100% overload. This factor, k, will necessarily be a constant for all members of this class. The impact, I, will be $\frac{165}{l+150}$ for railroad bridges.

Members subjected to dead- and live-load stresses of the same or opposite character, and in which the live-load stress is produced by a partial loading of the span, as for web members generally, shall be proportioned for the algebraic sum of the dead- and live-load stresses plus impact, using a unit stress, kf, where f is the allowable unit stress in tension or compression, as the case requires. Where a reversal in live-load stress is possible, the sectional area should be computed separately for each combination with the dead-load stress, and the larger area will govern the design.

A structure designed in this way will have a minimum factor of safety of 2 on the elastic limit for the assumed live load, and will be uniformly safe for an overload of 100%, in the live load. The unit stresses for the overloaded condition will thus approach the elastic limit for short spans wherein the dead-load stresses are small compared with the live-load stresses. The following example will illustrate the method.

Example.—Given, a single-track span of 452 ft., with 16 panels. (Intermediate span of the Memphis Bridge.) Compressive stresses are negative.

Top chord, $U_6\ U_8$

Dead-load stress	=	— 1 200 800 lb.
Live-load stress	=	— 889 500 "
$I = 0.275$.....	=	— 244 500 "
Total s.....	=	— 2 334 800 lb.

$$\text{Area required} .. = \frac{2\,334\,800}{15\,650} = 149.3 \text{ sq. in.}$$

$$f' = \frac{3\,468\,800}{149.3} = -23\,260 \text{ lb.}$$

$$k = \frac{f'}{f} = \frac{23\,260}{15\,650} = 1.485.$$

Diagonal, $U_8\ M_7$

Dead-load stress	=	— 20 100 lb.
Live-load stress	=	— 111 900 "
$I = 0.408$.....	=	— 45 700 "
Total s.....	=	— 177 700 lb.

Mr. Molitor.

$$\frac{s}{f} = \frac{177\,700}{10\,675} = 16.64 \text{ sq. in.}$$

$$f' = k\,f = 1.485 \times 10\,675 = 15\,850 \text{ lb.}$$

$$\text{Area required} = \frac{335\,300}{15\,850} = 21.15 \text{ sq. in.}$$

$$\frac{335\,300}{16.64} = 20\,150 \text{ lb.} = 1.89\,f$$

This shows that, for double the live-load stress, the unit stress in the top chord was increased 1.485 times; and, for the post, $U_8\,M_7$, the same overload would have increased the unit stress 1.89 times on the basis of 16.54 sq. in. of area. The actual area required by the foregoing specification is 21.15 sq. in. This same post also receives a counter stress in tension equal to the compression, but, as the dead-load stress is compressive, the counter stress did not give the maximum gross area.

Returning to the question of fatigue, which is merely a phenomenon of over-stress, and not particularly due to repetition of stress, the writer quotes the following:*

"*Fatigue of the Material.*—Based on the classic experiments of Wöhler (1859-1870) which were continued by Professor J. Bauschinger, a formula was proposed by Launhardt (1873) and later modified by Weyrauch, aiming to apply the principles of the fatigue of material to the design of bridge members.

"Wöhler's law states that for a large number of repeated load applications, rupture of a material is produced under stress which is below the ordinary (static) breaking strength of that material. The conditions under which Wöhler's experiments were made differ so radically from those attending the actual operating conditions of bridges, that it is questionable whether any one is justified in applying the Launhardt-Weyrauch formulæ to bridge designs.

"Wöhler's load repetitions followed in quick succession and were continued without interruption (several million times) until failure was produced. Bridge members are subjected to a repetition of stress which is always followed by a rather long period of rest, and few structures, even under the heaviest traffic, are retained in service long enough to receive several million applications of the moving load. Also, a well-designed bridge is never calculated for stresses approaching even the elastic limit, while Wöhler bases all his observations on stresses exceeding this limit.

"In addition to these facts, modern experiments made under conditions which correspond quite closely with bridge practice, though limited in extent, point uniformly against the existence of fatigue in bridges.

"The above mentioned Launhardt-Weyrauch formulæ have been extensively used in designing bridge members and are still retained in

* From "Kinetic Theory of Engineering Structures," published 1910, p. 265.

many specifications. However, this practice does not seem justifiable, especially when allowance is made for impact and secondary stresses generally. Mr. Molitor.

"The fatigue formulæ undoubtedly served a good purpose in the days when so many important factors were neglected, but at present the aim should be to make proper allowance for all the known stress elements and thereby reduce the 'factor of ignorance' to a minimum."

One hundred trains per day would give 3 600 000 load applications in 100 years; and a sample of wrought iron required 10 150 000 repetitions of a load of 35 000 lb. to produce failure.

As material undergoes a permanent set when stressed above the elastic limit, it is quite probable that such an excessive stress, if maintained sufficiently long, will produce failure. The great number of repetitions necessary to produce failure is undoubtedly due to the very short duration of each stress application.

Of far greater importance is the uniformity of bridge steel, which in the present state leaves room for much improvement. The writer refers particularly to such defects as piping and segregation, and those who may be interested in this matter are referred to his discussion of the paper by J. A. L. Waddell, M. Am. Soc. C. E., entitled "The Possibilities in Bridge Construction by the Use of High-Alloy Steels."*

CLEMENT E. CHASE,† JUN. AM. SOC. C. E. (by letter).—Mr. Prichard's long experience with the practical manipulation and use of iron and steel, and his very thorough knowledge of the results and significance of past tests on these metals, have given him an unusual insight into their nature. It is because steel is the most important engineering material of construction and because this paper deals with the fundamentals governing its behavior in use, that it is so valuable a contribution. There is a great multiplication of tests in these days, but too often they only serve to pile up a mass of loosely connected, superficial data. Lacking a framework of sound theory to which to adjust the test data, much of the value must be wasted. Long before the entire truth is known (if it ever is) about any subject, it is possible to establish a working theory, in the light of which the results of each experimentation should be examined and the theory added to, strengthened, or modified accordingly. Mr. Chase.

The theory underlying heat action on metals is in common use in America to check and explain in this way the results of all heat treatments, but, to the writer's knowledge, this is practically the first reference in American engineering literature to the theory of strain action, or what is called the "mechanism of plastic deformation." The importance of this theory to the engineer can hardly be over-estimated. Its development was due mostly to a small group of English physicists,

* *Transactions*, Am. Soc. C. E., Vol. LXXVIII, 1915, p. 1.

† Pittsburgh, Pa.

Mr. Chase.

Sir Alfred Ewing, Rosenhain, Humfrey, and Beilby, whose work in this line has only lately begun to be appreciated at its true worth. Their investigations led them into a hitherto unknown field—on the borderland between pure science and practical engineering—opened up by the application of the microscope to the examination of metallic structure. The first paper by Ewing and Rosenhain, announcing the discovery of the basic phenomena of the "slip bands", was published in 1900.* They then advanced the proposition that deformation under strain took place by slippage in each crystalline grain, along the cleavage planes, much as a pack of cards can be distorted without changing the shape of the individual cards. This view of the significance of the slip bands was attacked by continental scientists, and was defended in a series of able papers.† No one who reads these can help but have a feeling of great admiration for the resourcefulness, straight-thinking, and experimental skill with which these men proved their contentions and upheld their important theory. Dr. Rosenhain, who is now Superintendent of the Metallurgy Department of the National Physical Laboratory of Great Britain, last year published an excellent book in which this work is summarized.‡ Recently, American metallurgists have devoted some space in their books to the effect of strain on structure, and a new book§ by Professor Henry M. Howe, of Columbia University, devotes one of two parts to this branch of the science of metallography. In his preface, Dr. Howe says,

"The usefulness of steel really results from its resistance to deformation and its power to endure limited plastic deformation. Hence a knowledge of the mechanism of this deformation and of the way in which steel in part resists deformation and in part accommodates itself to it, may in time disclose the essence of its power of resistance and accommodation. To understand this essence is to be the better prepared to approach the problem of fitting the metal for service, not empirically alone, but also scientifically."

In the writer's opinion, this study is not one of which the metallurgists should be allowed to have any monopoly. Understanding and use of it will be of even greater value to engineers. Mr. Prichard approaches the matter from the attitude of the structural engineer. Following his thought, one is led into the subject from the familiar direction of physical tests of metal. The English metallurgists reached their conclusions largely from what they saw in the microscope. Mr. Prichard has come to the same point through searching for the significance of engineering test phenomena. It has been the writer's privilege to follow to some extent the preparation of this paper, and he regards it as worthy of record that Mr. Prichard's theory was

* *Philosophical Transactions*, Royal Society, Vol. 353A.

† *Journal*, Iron and Steel Institute, 1904, No. 1, pp. 335–390; 1906, Vol. 2, p. 189.

‡ "An Introduction to the Study of Physical Metallurgy", Van Nostrand.

§ "The Metallography of Steel and Cast Iron".

formulated, practically in its present shape, before the work of Beilby on the mobile and amorphous states of metal was known to him. The significance of this is that it shows how thorough study of the problem from either the engineering test direction or the micro-metallurgical direction leads to the same conclusion. Mr. Chase.

Mr. Prichard states that the deformation of metals in tension and compression is in reality caused by failure to resist shear. This conclusion comes irresistibly from a study of the microscopic phenomena of slippage, which is essentially a shear action. It must also be the conclusion of any one who has watched thoughtfully the formation of the lines, referred to by Mr. Prichard, on the mill scale of tensile tests. In regard to these lines, the writer cannot agree that they usually take a direction of about 26° 34′ to a plane normal to the direction of pull. It has been his observation that this angle is more nearly 45 degrees. Tensile tests of hooping and flat steel, however, fracture usually at an angle which is about 26° to this plane. Mr. Prichard's hypothesis, on pages 1437-38, as to the manner in which the particles in the crystalline grains slip, gives a simple picture of the action. Actually, the slips in the grains occur in any direction, depending on the orientation of the grain to the shearing stress and the location of the most favorably disposed cleavage planes, and in at least as many as three directions in the same grain under severe overstrain.

Fig. 3, representing the necking down of a tensile test piece, is subject to the criticism that the actual necking of test pieces gives much more support than this figure does to the conception that the failure is by shear in ductile metals. Fig. 10 shows the tension fracture of a test bar of structural steel. The necking of the piece was parallel to the line, *S-S*, which is also parallel to the fracture. This line makes an angle of about 28° 40′ with the axis of the test piece. It will be noted that slippage occurred along the line of fracture, or the zone of fracture, to such an extent that one-half of the test is displaced $\frac{1}{16}$ in. to one side. Fractures exactly similar to this have been noted in full-sized eye-bar tests. The more symmetrical "cup" fractures of tension bars also give evidence of this failure by shear.

The writer agrees with Mr. Prichard in what he says as to the elastic limit and yield point, and believes that there would be less confusion surrounding these terms if there was more appreciation of the significance of the phenomena of plastic deformation that this paper deals with. We now have "true" elastic limit, proportional elastic limit and yield point—and others. It seems to the writer very probable that there is no "first permanent set" without a corresponding variation from proportionality—and that the two limits are, in fact, coincident. It is only the fact that delicate extensometers can detect slight permanent sets at a lower point (often due to initial strains)

Mr. Chase.

than the eye can determine any regular breaking away of the stress-strain curve, that gives these two points apparently different values. The writer believes Mr. Prichard is right in saying that it is only that elastic limit which is at the beginning of the general yield that has any real significance. To indicate the method of locating the limit, the qualifying words, "by set", or, "by proportionality", might be used.

Clear light has been thrown on the long-debated question of "crystallization" and fatigue by the men to whose work Mr. Prichard refers on pages 1452 and 1455. The misunderstanding, between those who believe in "crystallization" of metal in service and those who do not, is mainly one of terms. "Crystallization" would have it that metal becomes coarsely crystalline under vibration or long-continued severe

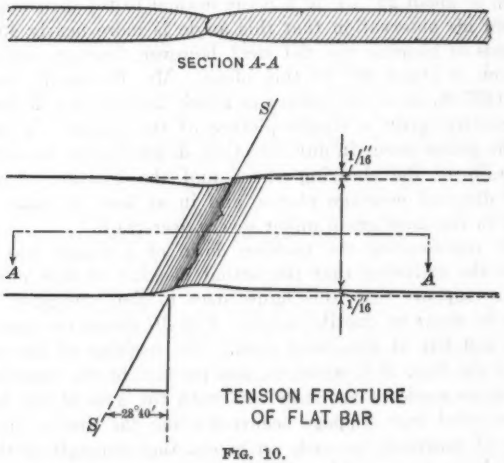


FIG. 10.

service—and points to the fracture of the metal in proof. Now, it is not true that metal becomes crystalline, for it always is crystalline—but it is true that under stresses approaching the elastic limit if repeated, or half that limit if alternated, an internal change takes place in the metal that eventually manifests itself by failure. That change is the development of internal slips into internal cracks, and results finally in a sudden snapping of the metal, with no showing of ductility, and with a fracture that satisfies the eye as crystalline. The deterioration that the layman insists on is true, if not as easily caused as he imagines —the theory as to its mechanism, that he implies in the term "crystallization", is not true. No indication has ever been found of any growth of the crystal size of steel at atmospheric temperature. However, for

those who declare offhand that the idea of crystal growth in cold steel is preposterous, there may be food for thought in the fact, which Mr. Prichard has mentioned, that in lead such crystal growth does occur under certain conditions at atmospheric temperatures. The microscope gives an explanation of the comparatively coarse size of the facets often found in a fatigue fracture. It is the progressive spread of the internal cracks in a plane through several crystalline grains while the deterioration is in progress. Mr. Chase.

HENRY S. PRICHARD,* M. AM. SOC. C. E. (by letter).—Mr. Chase, after describing the fine achievements of the English physical metallurgists, states: Mr. Prichard.

"In the writer's opinion, this study is not one of which the metallurgists should be allowed to have any monopoly. Understanding and use of it will be of even greater value to engineers. Mr. Prichard approaches the matter from the attitude of the structural engineer. Following his thought, one is led into the subject from the familiar direction of physical tests of metal. The English metallurgists reached their conclusions largely from what they saw in the microscope. Mr. Prichard has come to the same point through searching for the significance of engineering test phenomena."

In studying the phenomenon of plastic deformation of steel and iron, not through the eyepiece of the microscope, but by applying mechanical principles to the ordinary physical tests, the writer conceived of the flow by which the deformation is accomplished as consisting of the sliding relatively to each other of perfectly elastic solid particles separated by thin highly viscous mobile or liquid films. The investigations of the English physical metallurgists, as described by Messrs. Chase and Irwin, have led to precisely this conclusion. Under refined and ingenious methods of microscopical research with great magnifications the little masses or grains, as they are called, which are disclosed by ordinary microscopical examination, are shown to be compound, and have been resolved into numerous crystalline elements which are in themselves little crystals, and constitute the fundamental grains of the solid portions of the metal, which slide relatively to each other during plastic deformation.

As to the theory formulated by Mr. Irwin, and attributed by him to the writer, to the effect that these aforementioned compound grains remain perfectly elastic during plastic deformation, the writer had not the slightest intention of promulgating any such idea; on the contrary, it may be noted, incidentally, that, in referring, on page 1437, to what is seen under the microscope, he did not state that the slips between the sliding particles appeared as sharply defined lines

* Pittsburgh, Pa.

Mr. Prichard.

between the grains, but as sharply defined lines on the polished surface of each grain. The writer, however, was dealing primarily with the principles involved and not with the precise inner structure of the metal. It was not necessary, and he did not choose, to complicate the application of the principles by introducing details of structure; but he hoped that the details of structure would be discussed, and he is pleased that this phase of the subject has since been so ably handled by Mr. Irwin.

The current theory of plastic deformation included in the paper is useful in explaining the effects of over-straining iron and steel, and is destined to become of service in guiding the course of future experimentation.

The theory, as Mr. Chase explains, is not very old, and has only recently been presented to American engineers. That there should be some criticism and opposition is natural.

Phases of the theory have been investigated independently from different standpoints. Writers on theoretical mechanics have proposed three theories of the breakdown of the elastic resistance of an isotropic solid, as follows:

First.—That the elastic limit is reached at a certain intensity of direct stress;

Second.—That the elastic limit is reached at a certain intensity of linear strain;

Third.—That the elastic limit is reached at a certain intensity of shear. This was proposed by Mr. J. J. Guest,* and is the latest of the three theories.

Mr. Moore states that the paper quotes Mr. Hancock's tests as a proof of the maximum shear theory, and questions whether they are a confirmation. The word "proof" is a little too strong. The paper, in referring to plastic materials, made the statement:

"That shear is the critical consideration is supported by experiments and analysis of J. J. Guest, Professor E. L. Hancock, and others."

The comparative results of Hancock's tests† (of steel) are as given in Table 8.

After considering the results of many tests, the writer reached the conclusion, held by most authorities, that brittle or comminuible materials, under simple compressive loads, fail by shearing on planes at certain definite angles with the direction of the load; and, in comparing the theoretical and the actual planes on which slipping oc-

* *Phil. Mag.*, July, 1900.

† *Proceedings*, Am. Soc. for Testing Materials, Vol. VI, 1906, p. 305; and Vol. VII, 1907, p. 271.

curred, he remembered that slipping is resisted by friction (a feature on which Mr. Jonson lays stress, page 1480, and followed Navier's analysis.* Mr. Prichard.

TABLE 8.—PERCENTAGES WHICH THE MAXIMUM STRESS, STRAIN, AND SHEAR FROM TENSION OR COMPRESSION COMBINED WITH TORSION IN VARIOUS PROPORTIONS, ARE OF THE MAXIMUM STRESS, STRAIN, AND SHEAR FROM TENSION OR COMPRESSION ALONE.

Computed from Hancock's observations at the elastic limits, on the assumptions that these limits were well defined, that the material for each set of tests was uniform, perfectly elastic, isotropic, and free from initial stresses, and that Poisson's ratio (which affects strain only) was one-third.

Number of tests averaged.	Proportions of average tension or compression and average shear from torsion.	Maximum stress. Percentage.	Maximum strain. Percentage.	Maximum shear. Percentage.
	Criterion by each theory for all cases.....	100	100	100
Tubes, 1 in. outer diameter, $\frac{3}{4}$ to $\frac{1}{4}$ in. thickness (t).				
3	(Shear + Tension or Compression) = 0.120	97.9	98.8	99.2
8	do = 0.280	82.3	84.0	100.8
4	do = 0.610	78.8	84.7	96.4
5	do = 1.017	71.3	81.2	99.8
(t = ¼ in.) 1	do = 1.500	83.3	97.7	126.6
5	Shear only (from Torsion).....	55.7	74.3	111.4
Solid Sections.				
4	(Shear + Tension or Compression) = 0.220	93.8	96.2	99.2
3	do = 0.397	97.8	101.5	107.6
6	do = 0.613	100.5	108.6	124.8
2	do = 0.942	100.8	115.5	129.6
3	do = 1.880	120.1	145.7	196.8
3	Shear only (from Torsion).....	74.3	99.0	148.6

The writer reached the conclusion that plastic materials under simple tension or compression slide on slippage or gliding planes, from a study of the phenomena displayed in tests and, later, from Sir J. Alfred Ewing's account of his microscopical examinations. The conclusion that it was the shearing components of the simple tension or compression, acting on planes of maximum shear and uninfluenced by the component normal to these planes, was drawn from the fact that the elastic limit is the same in both tension and compression. Hancock's experiments on tubes support the theory that shear is critical. His experiments on solid sections seem to

* As given in Johnson's "Materials of Construction", p. 24.

Mr. Prichard.

favor the theory that stress is critical, but, in the writer's judgment, the computed results are much in error and untrustworthy. For solid sections, in such cases, the elastic limit is not well defined, and this and other departures from the assumptions tend to indicate stresses and shears much higher than the actual.

The writer's conception of shear is by no means "peculiar", as Mr. Jonson suggests. On the contrary, the conception of shear, as given on page 1436, is so orthodox that it can be found in any textbook on mechanics.

It may be well at this point to define shear:

"An action or force which causes or tends to cause two contiguous parts of a body to slide relatively to each other in a direction parallel to their plane of contact" (Webster).

Before the elastic limit is reached, shear only tends to cause contiguous parts of a body to slide relatively to each other.

According to Mr. Jonson, flow begins when the shear reaches the elastic limit, and flow takes place "only when the shear exceeds the elastic limit." This is in exact accord with the paper (page 1440).

The important differences between Mr. Jonson and the writer are in regard to what takes place beyond the elastic limit, and are two in number:

First, the writer contends that, beyond the elastic limit, the shear causes "contiguous parts of a body to slide relatively to each other in a direction parallel to their plane of contact," and that flow is the combination of a multitude of such slidings of numerous small parts accompanied by such rotations of the parts as may be necessary to continuity and alignment; but Mr. Jonson contends that the "use of the words 'sliding' and 'slipping' in connection with flow seems to be unfortunate, because flow is something entirely different from slipping. Flow consists of an indefinitely extended, angular deformation".

Second, the writer contends that part of the work done by the load after the elastic limit is reached consists in liquefying films of metal between the sliding surfaces, or, as Beilby puts it, conferring on them by purely mechanical movement the "mobility of the liquid state" (pages 1440 and 1460); Mr. Jonson, on the other hand, states:

"The author seems to conceive of the flow which takes place in ductile metals when stressed beyond the yield point as if it were due to a change in the metal from the solid to the liquid state. The energy required to bring about this change of state he seems to regard as a part of the work done by the load. This, however, is not in accordance with the facts. All the work done by the load previous to the beginning of flow is stored in the test piece as elastic energy."

Mr. Chase, after referring to the fact that deformation of plastic metals in tension and compression is caused by failure to resist shear, states: Mr. Prichard.

"This conclusion comes irresistibly from a study of the microscopic phenomena of slippage, which is essentially a shear action. It must also be the conclusion of any one who has watched thoughtfully the formation of lines, referred to by Mr. Prichard, on the mill scale of tensile tests."

(Incidentally, it may be stated that Mr. Chase considers that for these lines the angle of 26° 34′ to a plane normal to the direction of pull is too small. This angle was given in the paper as a rough approximation obtained by averaging the tangents of 0° and 45 degrees. A slippage plane, at an angle of 45° with a plane normal to the direction of the force, may make any angle between 0° and 45° with the surface of the test piece. The writer, since the paper was presented, has computed the average of the angles cut out by all possible intersections as nearly 32½ degrees.)

The facts discovered by the "small group of English physicists", to whom the development of the theory of the "Mechanism of plastic deformation" is mainly due, are so pertinent that it may be well to let these physicists tell the story. Mr. Beilby's account of what followed "The discovery that layers of a solid many hundreds of molecules in thickness can have a mobility of the liquid state conferred on them by purely mechanical movement" is quoted in the paper (page 1460).

Sir J. Alfred Ewing's account of the slips which take place within the crystalline grains of metal after the elastic limit is passed, is taken from a copy of the 1912 May Lecture before the Institute of Metals on "The Inner Structure of Simple Metals", with which the author recently favored the writer. The photo-micrographs referred to are reproduced by the authority of the Institute of Metals and Sir J. Alfred Ewing.

"My intention to-night is first to remind you briefly of some of the things that the microscope has taught us regarding simple metals, and then to go on to some more or less speculative considerations based on that knowledge. I propose to confine myself definitely to simple metals—that is, metals which behave as pure metals behave, leaving untouched the large and complicated subject of the alloys. Alloys present complexities that would only distract attention from the particular points which I wish to bring before you. Without those complexities we shall find the matter difficult enough.

"Some apology is due to the experts in the audience for presenting facts with which they are already familiar, but probably the audience includes some who are not experts, and in any case it is convenient to recapitulate a little of our positive knowledge before entering the region of speculation.

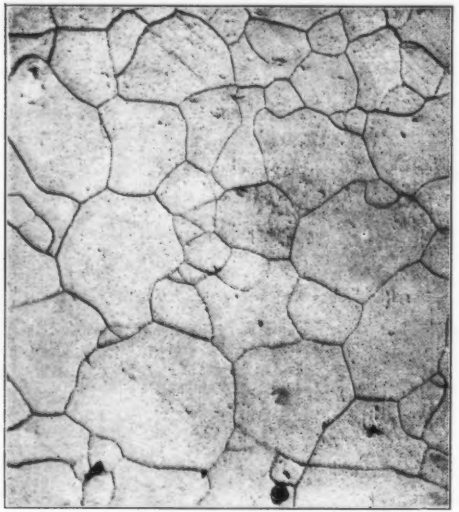
Mr. Prichard.

"Generally speaking, when we wish to apply the microscope to the examination of metallic structure, we begin by polishing the surface of the metal so as to remove those inequalities which would embarrass the use of the microscope, especially at high powers. As Dr. Beilby showed in the admirable May Lecture he gave last year, the process of polishing itself affects the constitution. It makes the metal on and near the surface entirely different in character from the metal within. It produces, according to his view—which, I think, is now generally accepted—an amorphous layer in contradistinction to the crystalline structure which, as we shall presently see, is revealed when that amorphous layer is removed. Consequently, what we have first to do after polishing is to remove the superficial layer before we can really see the normal characteristics of the structure, and the usual manner in which that layer is removed is by a light chemical attack, a slight etching with an acid or some other substance. Occasionally, we may resort to other means of removing the layer. An interesting method which is sometimes available is to heat the metal sufficiently to make the surface layer sublime away.

"When we lightly etch the surface of a metal, what do we see? In general we see an appearance such as is illustrated in Fig. 1, Plate I. This is a photomicrograph taken by Dr. Rosenhain of part of the surface of a bar of Swedish iron, magnified 150 diameters. A very similar appearance is presented by other metals. The surface is seen to be composed of a large number of separate grains, irregular both in size and shape, and also irregular as regards the character of the boundaries. Sometimes these are straight and sometimes curved. The shapes of the grains are as irregular as the counties in a map of England; their boundaries depend, like those of the counties, on historical conditions, as we shall see by and by.

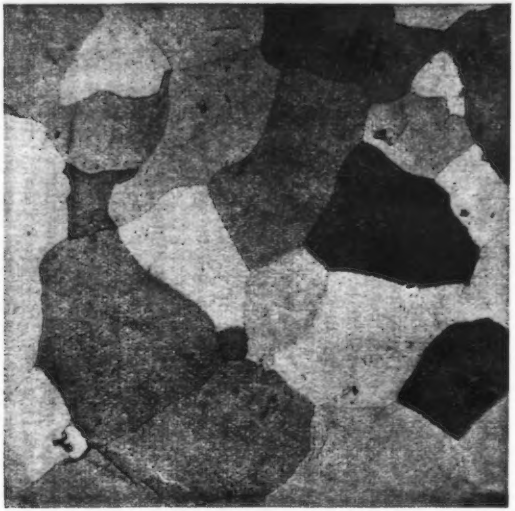
"Suppose now that we carry the etching a little further (Fig 2, Plate II). We discover that the grains can be distinguished not merely by these irregular boundaries. A difference of texture begins to manifest itself between one grain and another; some grains are very bright, some are more or less dark, and some are very dark. If instead of illuminating the surface directly from above, as is the case in Fig. 2, we throw the light from one side, we discover that the same grain which appears bright under one condition of illumination becomes dark under another. Compare Fig. 3, Plate II, with Fig. 2.

"These are two photographs, for which I am indebted to Dr. Rosenhain, showing the same part of the surface of one metal under two different conditions of illumination. In Fig. 2 the light comes directly from above; it strikes the surface perpendicularly, and is reflected up into the microscope. In Fig. 3 the same grains are illuminated by light coming from one side. It will be observed that there is no difficulty in identifying the same grains in both; and that grains which are very bright under the first illumination become dark under the other. If we move the source of light to another side, or turn the specimen round, so as to alter the direction from which the light falls upon it, we find the grains vary in brightness in the most remarkable manner, sometimes flashing out brilliantly and sometimes becoming almost entirely dark. This is true of all metals. These

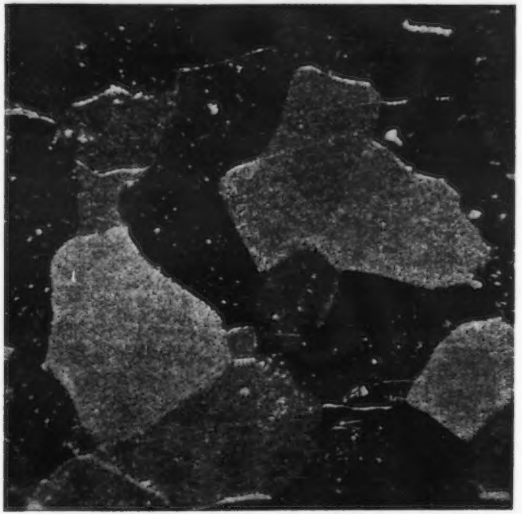


"Fig. 1, Plate I.—Bar Swedish Iron as Rolled. Magnified 150 diameters."

"Fig. 10, Plate I.—Same surface as Fig. 1 after straining in tension, showing slip bands."



"FIG. 2, PLATE II.—NEARLY PURE IRON ANNEALED IN HYDROGEN AND DEEPLY ETCHED. MAGNIFIED 100 DIAMETERS. VERTICAL ILLUMINATION. (ROSENHAIN.)"



"FIG. 3. PLATE II.—SAME SURFACE AS FIG. 2, BUT WITH OBLIQUE ILLUMINATION."

photographs are of iron, but we find gold, silver, copper, lead, and so forth, exhibiting precisely the same general characteristics. Mr. Prichard.

"Examination of the etched surface under a high power shows that this difference of texture is really due to a multitude of little facets or tiny plane surfaces in each grain which are causing the general surface of the grain to reflect light in a particular manner. They are acting like a multitude of little mirrors all facing one way. These facets are parallel in any one grain, but have different inclinations in the different grains.

"In Fig. 4, Plate III, we have a photograph of a grain of iron, not very deeply etched. A part of a single grain occupies nearly all the photograph under a power of 800 diameters. You observe that over the grain, here and there, are a number of pits which are clearly geometrical in form; they happen to be nearly square in this particular case. In some places a number of pits have run together forming a black irregular patch, but in other places you can see the individual pits quite clearly. These pits are formed in the process of etching.

"Under more favourable conditions, with deeper etching, the whole surface becomes covered with such pits. Fig. 5, Plate III, is a photograph published a good many years ago by Dr. Stead, which shows very clearly what it is that gives rise to what I have called the texture of the grain. In the former example you had only isolated pits, but in this one the appearance is such as would be presented if we were to take a great mass of brick work and pick out the superficial bricks all over it, so as to reveal the character of the structure as built up of brickbats. That, in effect, is what happens in the etching of a metal.

"I do not know any example which gives a clearer indication than this does of what causes the difference of texture in the surface of these grains, nor one that indicates more plainly the real nature of their structure, as developed by etching. One can see unmistakably how the surface of the grain consists of a multitude of geometrically similar pieces, parallel to one another, so that their corresponding facets are all oriented one way. They are oriented in different ways as we pass from grain to grain, but in any one grain they face one way, and in consequence of that the light which falls on the grain is reflected in a perfectly uniform manner over the whole expanse of that grain, although it is reflected in a very different manner from the surface of any other grain. Over each grain the brightness is uniform, because the little surfaces are acting equally as regards the reflection of light.

"From this it is an easy step to infer that throughout the whole volume of any one grain there is an assemblage of pieces, which we may think of as the "brickbats" or structural units that build up the grain, all facing one way in the one grain, but facing different ways in different grains. Fig. 6, Plate IV, is another example, a piece of etched tin-plate, exhibiting the same characteristics. It shows a portion of two grains of the thin layer of tin with the boundary between them, and the difference of brightness is very marked. They are both exposed to the same light, but they reflect different amounts into the microscope. The reason is that the little facets on one are much more favourably directed for the purpose of reflecting the light back to the microscope than are the facets on the other.

Mr. Prichard.

"One might multiply illustrations all pointing to the same fact, namely, that the etching has revealed a definite geometrical structure within the grain by removing, as it were, a few of the superficial brickbats, leaving cavities and protuberances of a geometrical form. The conclusion is simply this, that every one of these grains is in reality a crystal. Notwithstanding the irregularity of the boundary it has the true property of a crystal, the uniformity of internal structure which is the characteristic of a crystal. It is, to use Kelvin's phrase, a 'homogeneous assemblage' of structural units which is put together with greater regularity than any structure built up of definitely formed brickbats." * * *

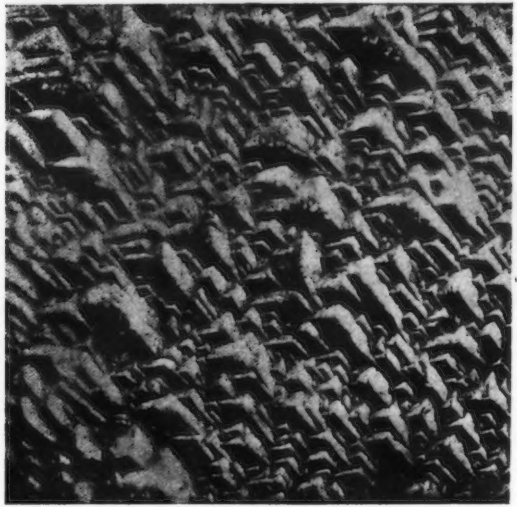
"A very important point about metallic structure is that we find true crystal grains not only in metals in the cast state but also in metals in other states, metals which have been wrought, which have been shaped by working even in the cold condition, and also in metals that have been worked in the cold condition and have afterwards been annealed by bringing them to such a temperature that a rearrangement of the grains has taken place. Whether we deal with them in the cold condition or in the annealed condition, we still find the same general characteristics, still the same granular structure, and still the same plain evidence that each grain is in reality a crystal.

"Take, for instance, a bar that has been shaped by being passed through a rolling-mill in the cold state. One of the photographs already shown (Fig. 4, Plate III) is part of the transverse section of a cold-rolled iron bar, rolled down from a comparatively large diameter, so that the individual grains within the bar have suffered tremendous distortion in the process of rolling. The greater part of the field is covered by a single crystal. Over its whole surface there are geometrical pits. When it was examined very carefully in the research by Dr. Rosenhain and myself we found that these pits were parallel all over the crystal notwithstanding the tremendous distortion it had undergone. It was clear, therefore, that the regular parallel grouping of the structural units or brickbats had in some way or other been preserved during the process of severe straining.

"The point is further illustrated if you examine in the microscope a specimen of metal that has been broken in a testing machine. Take a fairly plastic metal, such as iron or mild steel, which stretches a good deal before it breaks. If you polish and etch the side of the bar near the fracture, where a considerable amount of extension has taken place, you will find that the metal there consists of grains similar to those you have already seen, but with this difference, that these grains are all elongated in the direction of the stretching. Their shapes may be still very irregular, but there is clearly a predominating greater length in the direction along the bar, as compared with the transverse direction. The stretching the specimen underwent before it broke has elongated each grain, but its granular character persists.

"The crystalline constitution of the grains, then, survives severe straining. How does it do so? That is a question Dr. Rosenhain and I set ourselves to answer. We tested specimens of metal by straining them actually under the lens of the microscope and observing what happened during the process. The specimen was a thin strip of sheet metal, which was strained in such a manner that the same crystals

"FIG. 4, PLATE III.—IRON, SHOWING PITS PRODUCED BY ETCHING. MAGNIFIED 800 DIAMETERS."

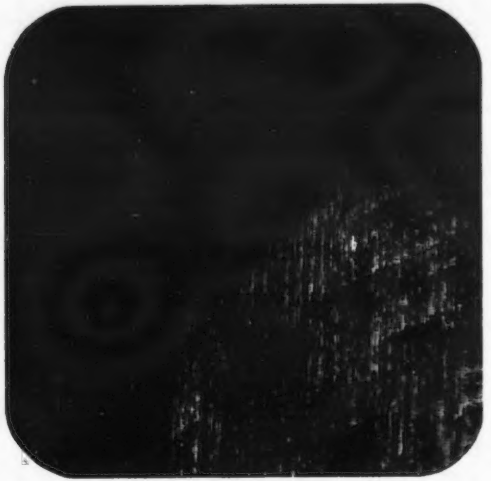


"FIG. 5, PLATE III.—SILICON STEEL. MAGNIFIED 130 DIAMETERS. (J. E. STEAD.)"

"Fig. 6, Plate IV.—Tin plate, showing boundary between two grains of the tin. Magnified 100 diameters."

"Fig. 15, Plate VI.—Slip bands in strained lead. Magnified 1 000 diameters."

"FIG. 13, PLATE VII.—LEAD AFTER STRAINING, WITH SLIP BANDS SHOWN BY OBLIQUE LIGHTING. MAGNIFIED 100 DIAMETERS."



"FIG. 14, PLATE VII.—SAME SURFACE AS FIG. 13, BUT WITH DIRECTION OF LIGHTING ALTERED TO SHOW SLIP BANDS ON A NEIGHBOURING GRAIN."

4 PAPE

were kept in view the whole time. In Fig. 1, Plate I, we had a specimen of iron lightly etched. This was subsequently strained by tension, but the photograph of Fig. 1 was taken before the straining began. In Fig. 10, Plate I, we have identically the same grains after a slight amount of straining—enough to carry it beyond the elastic limit, but not much beyond. If we compare those two accurately by applying compasses and measuring the lengths of the grains, it will be found that in Fig. 10 each grain has become a little stretched in one direction and a little shortened in the transverse direction. But that is not the main difference. The main difference produced by the straining is that over the surface of each grain a number of curious black lines have appeared, almost like the crevasses of a glacier, lines which are substantially straight and substantially parallel. It is in virtue of these lines that the plastic strain of the crystal grain has happened. These lines mean not that crevasses are formed, for there is no rupture of continuity, but that there has been shearing at a corresponding number of internal surfaces, that the crystal grain has behaved as a pack of cards behaves when you try to make it alter its form. The pack of cards becomes strained by the slipping of one card on the other, of each layer on its neighbor. In precisely the same way the crystals of metal become strained by the slipping of the little brickbats of one layer on those of the adjacent layer within each grain. The result is that on the polished and etched surface little steps are formed by the slipping, and it is these little steps that constitute the black lines you are now looking at. They are narrow bands rather than lines; Dr. Rosenhain and I, when first we discovered them, called them 'slip bands'." * * * Mr. Prichard.

"Clear proof that the slip lines are really little steps was furnished by testing the effect of oblique illumination. It will be obvious that, if the theory is correct, it should be possible, by throwing the light from the side, to get the little step which was dark in the first instance to shine up brightly. We have only to choose an appropriate direction from which the light should come in order that the step may reflect it up into the microscope. That has been done in this slide. Here are illustrations of what Dr. Rosenhain and I found when we made that experiment. In Fig. 13, Plate VII, we have several grains of a strained specimen of lead illuminated by light falling very obliquely from one side. The light is so placed that some of the slip lines or slip bands are bright, through reflection from the little steps up into the microscope. The light is falling on all the grains alike, but only one is visible, because none of the slip lines on the others are favourably situated for reflection into the microscope. Now we shift round the direction of the illumination (Fig. 14, Plate VII): another crystal has its slip lines brilliantly illuminated, and at the same time the one that had its lines illuminated before has now become dark.

"Fig. 15, Plate VI, is a photograph of the system of slip lines on a small part of a single lead crystal under a magnification of 1 000 diameters. You can see that the slips have produced small differences of level, and it is apparent that they have taken place successively in the different planes, so as to result in a compound system of steps.

"Dr. Rosenhain gave subsequently a further demonstration by obtaining a transverse section of the steps formed by slipping. To do

Mr. Prichard.

this he strained a piece of iron to form the lines, and then deposited a thick layer of copper upon it by an electrolytic process; finally he cut a transverse section through both the iron and the copper covering, and polished that for microscopic examination. Under a high power it was seen that the surface between the two metals had upon it a number of little definite parallel steps, which corresponded to the slip lines produced by straining the iron.

"We conclude, then, that the plastic yielding of metals under strain is due to slips occurring on the gliding planes of the crystal grains. This notion gives a key to plasticity in metals. It is not going too far to say that any amount of distortion can be accounted for by slips of this kind without requiring the continuity of the crystalline structure to be interrupted."

The continuation of this story of the "Mechanism of plastic deformation" is by Dr. Rosenhain:

"If we realise the true nature of this phenomenon of deformation by slip, we see at once that it throws a flood of light on the behaviour of metals under stresses sufficient to bring about plastic strain. If such plastic strain occurs solely by slip, then the truly crystalline nature of the metal should remain unaltered by the straining process. We have already seen that strained, *i. e.*, elongated crystals, still exhibit the essential characteristics of crystals, so that, broadly speaking, this generalisation is correct. But there are a whole series of other phenomena connected with the process of plastic strain which would be extremely difficult to explain satisfactorily on any theory which required the structure of metal to remain perfectly crystalline, even when very severely strained. The circumstance that a metal is hardened by strain, in the sense of having a much higher yield-point and limit of elasticity, and even a higher breaking stress, is one of these facts. These circumstances have led the author to adopt a view put forward in the first place by Beilby [p. 100], to the effect that while plastic deformation—and although we have merely discussed its occurrence in connection with tensile strain, its nature and mechanism is the same whatever the system of forces which have brought it about—takes place by slip on the gliding planes of metallic crystals, yet that the act of slipping is accompanied by something further. Just as the rubbing action of polishing produces on metal surfaces a thin layer of altered, amorphous material, so we may well expect that the sliding over one another of adjacent slip surfaces will produce a local disturbance of molecular arrangement. If the slip is slight, then it probably happens that this derangement is also slight and temporary and that the disturbed molecules are still able to rearrange themselves pretty much in their original system. In such a case the crystalline orientation is not at all disturbed. If, on the other hand, the slip has been more pronounced, the resulting local disturbance will also be more far-reaching, a greater number of molecules will be disturbed, and they will no longer be able to re-arrange themselves in the old crystalline system. A more or less thin layer of amorphous metal will thus be formed on each surface of slip. At first—for a short time—these layers will probably possess a certain degree of mobility, like the surface film which adjusts itself under surface

tension. During this period these layers would act as a sort of lubricant, facilitating further slip on the same gliding planes. After a time, however, when the disturbed molecules have had time to 'set' in the amorphous condition, we should have on each plane where slip has taken place a layer of hard, non-plastic, amorphous metal. These would effectually prevent further slip on that particular set of gliding planes, and the crystals would be limited, in accommodating themselves to further plastic strains, to slip on other surfaces which had not been 'used' in the previous straining. But all the surfaces of *easiest* slip will have been used at the first straining, so that to effect plastic deformation the second time more force would be required. Not only this, but the hard and brittle amorphous films on the surfaces of previous slip would also act as a stiffening skeleton for the whole crystal and thus offer additional resistance to the commencement of fresh slip." * * * Mr. Prichard.

"If we regard cohesion within the body of a crystal as being due to the attractions between layers of adjacent molecules, we can readily understand the continuity of such cohesion acting throughout the entire mass of any one crystal. The forces which are at work in producing cohesion between adjacent crystals must, however, be of a somewhat different character, for it is obvious that the regular arrangement of molecules in oriented layers cannot be carried on through a crystal boundary, while it appears that the actual cohesion between adjacent crystals is stronger than that between different layers of the same crystal." * * *

"In addition to changing their own shapes, the crystals must move relatively to one another. The strength of the crystal boundaries is found to resist such movement, and the slip-bands are found to be arranged in such a way as to minimise the amount of displacement occurring at the actual boundaries. In other words, the metal takes up the new shape imposed on it, as far as possible, by means of slip within the crystals and with as little disturbance as possible of the inter-crystalline boundaries. That there is some definite movement at the boundaries becomes evident if a polished specimen is strained without being previously etched; the effect of the strain at once causes the crystal boundaries to become visible on the surface."*

Structural steel is a mixture in which more or less connected hard grains of an alloy termed pearlite are embedded in a softer matrix composed of grains of iron. The grains of pearlite consist of alternate thin layers of iron (Fe) and cementite (Fe_3C) in the proportion of six parts of iron to one part of cementite. If a specimen of such an alloy is

"provided with a polished surface and is then plastically strained, it will be found that the effect on its surface appearance is almost identical with that obtained by etching—the laminated structure is very clearly revealed. Close examination has shown that what really happens is that slip occurs along, or very close to, the boundaries of the

* "An Introduction to the Study of Physical Metallurgy", by Walter Rosenhain, Superintendent, Metallurgy Department, National Physical Laboratory, pp. 245–247, 257 and 258.

Mr. Prichard.

lamellæ of the two constituents present. When it is borne in mind that these lamellæ are formed by the process of crystallisation, it will be seen that their surfaces must lie on or near to some of the principal crystallographic planes, so that it is not really surprising that slip should occur on these surfaces."*

Grains of iron, as shown by photo-micrographs, are irregular as to size and shape, somewhat as in Fig. 11, except for the grain, *A-B-F-G*, which is drawn of conventional shape for simplicity of illustration. The slipping of the layers of the grain upward and to the right as a result of over-straining is shown in Fig. 12, which is drawn for the grain, *A-B-F-G*, enlarged. A second slippage upward and to the left will make offsets in the first slippage planes, as in Fig. 13. If the layers simply slipped, the grain, as a whole,

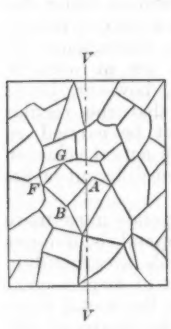


FIG. 11.

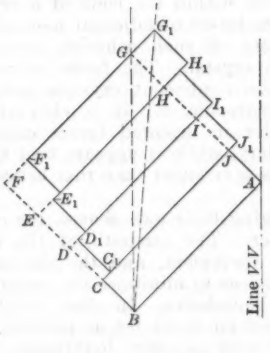


FIG. 12.

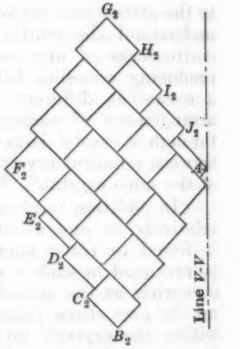


FIG. 13.

would tip to the right on the first slippage and crowd the neighboring grain adjacent to the line, *A-G*, and would tip back again on the second slippage. It is Professor Howe's opinion that the layers simply slip and project into the adjoining grains. He has discussed the matter at some length,† and concludes that the phenomena observed are those which would naturally follow in case there were no such rotation. It seems to the writer, however, that the geometrical necessities of the case require some slight rotation of the layers with each slip. The aggregate angle of rotation might not be great, however, as the rotations from slips in opposite directions tend to balance.

Professor Howe gives a valuable analysis of the effect of the rapid hardening of mobile metal during a test. It is pointed out in

* *Ibid*, p. 261.

† "Metallography of Steel and Cast Iron."

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the paper that such action may take place (page 1450). The writer does not consider that all the metal made mobile or viscous during a test will entirely solidify before the test is completed. Some portion may remain viscous for a while during a rest after being over-strained (pages 1450-51), and for a long while under repeated stresses. Professor Howe's suggestion, that a very important degree of true fatigue may occur, even before sub-microscopic tears form, seems to be in line with this thought and very reasonable.

That there is a temporary fatigue is shown by the behavior of iron and steel for a period after being over-strained. One manifestation is greater deformation under repeated stresses within the primitive elastic limit, as illustrated in Table 9, and under alternating stresses, as shown by a paper recently read before the Royal Society.*

The presence of some viscous metal on the planes of slippage and beneath the polished surface of the test piece would facilitate etching; and its subsequent solidification during a rest or gentle heating would make the test piece more difficult to etch, and thus explain "why after a heating so gentle as to intensify the hardness caused by deformation no traces of the amorphous metal along the slip planes can be detected."

"The explanation of the process of failure by 'fatigue', *i. e.*, under the repeated alternations of a stress which would not cause fracture if steadily applied, is also furnished by the conceptions described above as to the behaviour of a crystalline aggregate under strain. A stress which is to cause ultimate failure after repeated alternations must be large enough to produce a small amount of local yielding in the metal. This may be so small in amount as to be unobservable, even with a delicate extensometer, and in that case the stress would be regarded as lying within the apparent or 'primitive' elastic limit, but the microscopic examination of polished test-pieces under load has shown that the formation of slip-bands in isolated crystals here and there in the metal may and does occur for stresses of this kind. Some crystals, by their shape and the orientation of their gliding planes are unfavourably situated to resist the particular system of stresses which has been applied, and a slight local slip takes place.

* The following is a transcript of a printed abstract, from the paper, furnished to the writer by Professor W. C. Unwin:

"W. MASON. On Speed Effect and Recovery in Slow-Speed Alternating Stress Tests. Communicated by Professor B. Hopkinson, F. R. S."

Repeated cycles of equal direct and reverse torque have been applied to mild steel specimens of tubular form, and systematic measurements made of the range of the corresponding torsional strains. The speed of application of the cycles was varied between 2 and 200 per minute. After a considerable number of cycles at 2 per minute, the range of non-elastic strain was reduced about 50% on change of speed to 200 per minute, the range of torque being unaltered.

Similarly, if a specimen had endured a considerable number of cycles at 200 per minute, the range of non-elastic strain was immediately increased by 50 to 75% at change of speed to 2 per minute. If the latter speed was maintained, the augmented range of strain decreased, quickly at first, then more and more slowly. This recovery is compared with the reduction of range of strain due to a period of rest. The author attempts to account for these variations of strain on the hypothesis of alternate production and hardening of "mobile material in the steel."

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TABLE 9.—PROGRESSIVE INCREASE IN STRETCHING EFFECT OF SUCCESSIVE INCREASES IN LOAD (PULL) FROM 5 000 TO 40 000 LB. PER SQ. IN. ON A SPECIMEN, FROM A STEEL MORTAR HOOP, WHICH WAS SUCCESSIVELY STRAINED BEYOND ITS PRIMITIVE ELASTIC LIMIT OF 45 000 LB. PER SQ. IN. BEFORE THE LOAD WAS REDUCED AND AGAIN INCREASED.

Length of Specimen, 52½ in.; Gauged Length, 30 in. Compiled from Test No. 110, Watertown Arsenal Report, 1884, pp. 345-347.

INCREASE IN LOAD (PULL) PER SQUARE INCH.		FIRST LOADING.		Second loading. Elongation per inch, in inches.	Third loading. Elongation per inch, in inches.	Fourth loading. Elongation per inch, in inches.	Rested under initial load of 1 000 lb. per sq. in., 15 hours.	Fifth loading. Elongation per inch, in inches.	Sixth loading. Elongation per inch, in inches.
From:	To:	Permanent set, in inches.	Elongation per inch, in inches.						
5 000	10 000	0.0	0.000194	0.000193	0.000217	0.000210		0.000210	0.000200
10 000	15 000	0.0	0.000160	0.000177	0.000193	0.000200		0.000190	0.000220
15 000	20 000	0.0	0.000160	0.000176	0.000190	0.000197		0.000207	0.000196
20 000	25 000	0.0	0.000170	0.000187	0.000190	0.000203		0.000197	0.000194
25 000	30 000	0.0	0.000160	0.000187	0.000200	0.000200		0.000203	0.000226
30 000	35 000	0.0	0.000163	0.000186	0.000217	0.000210		0.000207	0.000204
35 000	40 000	0.0	0.000173	0.000200	0.000203	0.000224		0.000206	0.000223
Total of the above 5 000	40 000	0.0	0.001180	0.001306	0.001410	0.001444		0.001420	0.001463
40 000	45 000	0.000017	0.000177	0.000227	0.000240	0.000220		0.000227	0.000217
45 000	49 000	.....	0.001260	.....	.....	.....		.....	.....
45 000	50 000	.....	.....	0.000383	0.000250	0.000243		0.000230	0.000233
50 000	55 000	.....	.....	0.005194	.....	.....		.....	0.000250
50 000	56 000	.....	.....	.....	0.000720	.....		.....	.....
56 000	60 000	.....	.....	.....	0.004604	.....		.....	.....
55 000	60 000	.....	.....	.....	.....	.....		.....	0.000253
50 000	62 000	.....	.....	.....	.....	.....		0.002643	.....
60 000	65 000	.....	.....	.....	.....	.....		.....	0.000310
62 000	70 000	.....	.....	.....	.....	.....		0.011560	.....
65 000	72 000	.....	.....	.....	.....	.....		.....	0.003407
72 000	86 000	.....	.....	.....	.....	.....		.....	0.077300

Ultimate strength on sixth loading, 87 170 lb. per sq. in.

If the load remains in steady action, nothing further occurs. If, however, the stress is reversed—*i. e.*, if the metal is being subjected to alternating stress—then this slight amount of slip will also be reversed, particularly as the slip surfaces will still be covered with the temporarily mobile layer of amorphous metal. Such reversal will be repeated with each reversal of the applied stress, and at each successive slip the layer of amorphous material will be increased. After a time, however, by virtue of its temporary mobility, this film of quasi-liquid metal will be squeezed out between the gliding surfaces, and the site of the initial minute slips will develop into a fine crack. As this process continues, that particular crystal soon begins to lose its strength, and additional stress is thereby thrown upon its immediate neighbours, which undergo slip and gradual disintegration in the same way. Ultimately, the crack or flaw thus originated works

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its way across the entire section of the metal, and failure of the piece results. If the action has been fairly rapid, *i. e.*, if the stress was high enough to produce somewhat rapid disintegration by repeated reversals of slip, the resulting fracture exhibits the crystal faces upon which slip has taken place as a number of bright facets resembling those produced in a 'brittle' shock fracture, and it is this appearance which has led to the mistaken idea that alternating stresses cause metal to 'become crystalline'. Actually, as we have seen, the metal is crystalline from the beginning, never really 'fibrous', and the 'fibrous' or 'crystalline' appearance of the fractures depends on the mechanism of fracture, and not on any change in the crystalline structure of the metal.

"The explanation of fatigue fracture which has just been given, although it has been deduced from the general character of plastic deformation by slip, was first given by Ewing and Humfrey, as the result of direct experimental observations in which they watched the formation of slip-bands in certain crystals, and their gradual growth into cracks on the polished and etched surfaces of pieces of Swedish iron submitted to the Wöhler test."*

It is stated in the paper that experiments by Wöhler during the period, 1858-1870, indicate that the elastic limit can be raised when part of the stress is constant and part constantly repeated (page 1460); Launhardt devised a formula, following Wöhler's experiments (page 1461); but such formulas are not applicable to the design of bridge members, for the reason that the deformation beyond the yield point is ruinous (page 1462). In epitomizing these facts in the Synopsis (page 1433), the writer hoped to emphasize the caution against relying on any increase above the primitive elastic limit, in designing tension and compression members; and he did not desire to imply, by the statement quoted from the Synopsis by Mr. Stanton, that the strength can be raised above the yield point by repeated stresses "varying from zero to an upper limit."

Mr. Stanton states "that the phenomenon of repeated loadings of a structure from zero to an upper limit is only a particular case of cyclical variations of stress", and he refers to a "valuable paper" by Mr. Bairstow. Mr. Bairstow's conclusions in the paper referred to are as follows:

"It is found that, after a sufficient number of repetitions, iron or steel is capable of adjusting itself to variations of stress, cyclically applied. When this adjustment is complete, the specimen is found to have become perfectly elastic throughout the whole cycle, and fatigue does not occur.

"This adjustment to a given cycle is possible because the limits of elasticity are not fixed, but can be raised or lowered by repetitions of stress.

* "An Introduction to the Study of Physical Metallurgy", by Walter Rosenhain, pp. 253–254.

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"During the adjustment of the elastic limits to a given cycle of stress a change of length occurs in the specimens, which is the same as the extension observed in an ordinary tensile test when the yield stress is exceeded. For cyclically applied stress a similar extension occurs, even when the maximum stress in the cycle is less than the static yield stress.

"The greater the extension of the specimen during adjustment, the greater are the amounts by which the elastic limits are raised.

"The power of adjustment is limited, and if the range of stress in the imposed cycle is sufficiently great, the specimen becomes or remains inelastic, and work is performed during each cycle. This work is expended in moving portions of the crystals relatively to one another, and is probably associated with microscopic slip-lines which gradually develop into cracks, ultimately causing fracture of the specimen."*

In addition to the experiments on repeated stresses to which Mr. Stanton refers, Mr. Bairstow made some successive tests in tension and compression on axle steel (which had first been subjected to a cycle of alternate stresses of sufficient duration to secure equal elastic resistance of the metal to tension and compression), and obtained the following results:

"Following a final load of 18.6 tons per sq. in. in tension, the specimen was heated in boiling water for 15 minutes. Test No. 2 showed that the specimen was still elastic at the same load. In compression the limit was reached at about 8.5 tons per sq. in., and the load was continued to 13.28 tons per sq. in. Recovery was again produced, and it was then found that the elastic limit in compression had been raised at least to 13.28 tons per sq. in., but that the tensile elastic limit had fallen to 13.0 tons per sq. in. By alternate heating and testing, the elastic limits were moved about very considerably, but always with the condition that if the tension limit was raised the compression limit was depressed, and *vice versa*."†

Bauschinger and others have tested steel and iron alternately in tension and compression, with somewhat similar results. In the above quoted alternate tension and compression tests by Bairstow, the range of stress between the elastic limits in tension and compression remained nearly constant at about 27 tons per sq. in., against an original yield point of 24.9 tons per sq. in. On the other hand, some re-tests of eye-bar steel at the Watertown Arsenal, after a rest of 3 years and 3 months, had a range between elastic limits of from 82 000 to 98 000 lb. per sq. in., against a primitive elastic limit of 34 400 lb. per sq. in., as shown in Table 10.

After referring to the fact that in some cases raising the elastic limit by over-straining in tension reduces it in compression, Rosenhain states:

* "The Elastic Limits of Iron and Steel under Cyclical Variations of Stress", by Leonard Bairstow, *Phil. Trans.*, Royal Society, Vol. 210, p. 37.

† *Ibid*, p. 53.

"It is difficult to see how the formation of hard amorphous layers on surfaces of slip can account for a softening of the metal in relation to compression while producing hardening as against tension."* Mr. Prichard.

TABLE 10.—RE-TEST OF EYE-BAR STEEL.

Compiled from Watertown Arsenal Report, 1890, p. 731.

Original Eye-bar { Elastic Limit 34 400 lb. per sq. in. / Strained to 54 350 " " " "

Specimens from bar tested 3 years and 3 months later.

How specimens were taken.	UNANNEALED SPECIMENS. ELASTIC LIMIT PER SQUARE INCH.		ANNEALED SPECIMENS. ELASTIC LIMIT PER SQUARE INCH.	
	In tension.	In compression.	In tension.	In compression.
Crosswise.....	40 000	51 000	40 000	43 000
Diagonally.....	40 000	47 000	42 000	40 000
Lengthwise.....	63 000	35 000	47 000	42 000

It is not at all necessary to attribute paradoxical qualities to the "amorphous layers" to explain the softening of the metal in relation to compression which sometimes results from over-straining in tension, as it can be explained much better by the changes which occur in the balanced internal stresses whenever the metal is over-strained.

In 1848, James Thomson (the elder brother of Lord Kelvin), an engineer and eminent physicist, by a pure effort of scientific reasoning, and without any tests on the subject to guide him, deduced and published† the following theory:

"The considerations adduced seem to me to show clearly that there really exist *two elastic limits* for any material, between which the displacements or deflexions, or what may in general be termed the changes of form, must be confined, if we wish to avoid giving the material a set, or, in the case of variable strains, if we wish to avoid giving it a continuous succession of sets which would gradually bring about its destruction; that these two elastic limits are usually situated one on the one side and the other on the opposite side of the position which the material assumes when subject to no external strain, though they may be both on the same side of this position of relaxation; and that they may therefore with propriety be called the *superior* and the *inferior limit* of the change of form of the material for the particular arrangement which has been given to its particles; that these two limits are not *fixed* for any given material, but that, if the change of form be continued beyond either limit, two new limits will, by means of an alteration in the arrangement of the particles of the material, be given to it in place of those which it previously possessed."‡

* "An Introduction to the Study of Physical Metallurgy", p. 252.

† In an article in the *Cambridge and Dublin Mathematical Journal.*

‡ Encyclopædia Britannica, 9th Ed., Vol. VII, p. 800.

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Thomson illustrated his theory by demonstrating theoretically, for a particular case of torsion, that a bar, originally free from initial internal stress, could have its elastic resistance to torsion increased one-third in the direction of the straining force, and decreased one-third in the opposite direction, by a certain degree of over-straining; and he pointed out that the same principles are applicable to beams subjected to transverse loading. He did not allude to direct tension and compression, but his theory can readily be extended to such cases. For illustration: if a pair of soft steel eye-bars, with an elastic limit of 30 000 lb. per sq. in., and a pair of hard steel eye-bars of the same size, with an elastic limit of 50 000 lb. per sq. in., are symmetrically coupled in the same link by pins, and supported laterally, they can be alternately strained in tension and compression to 30 000 lb. per sq. in. without exceeding their elastic limits, provided they are free from flaws and initial internal stresses and have perfect elasticity to the elastic limit; thus showing the range of the elastic field for the link between the superior and inferior limits of its elasticity to be 60 000 lb. per sq. in. If, however, the link is strained in tension to the point where the tension in the hard steel bars is just below their elastic limit, the tension in the soft bars will be very little above 30 000 lb. per sq. in., and in the link as a whole it will be about 40 000 lb. per sq. in.; and, after the link is relieved of its load, the soft bars will be strained to 30 000 minus 40 000, or 10 000 lb. per sq. in. in compression, and the hard bars to 50 000 minus 40 000, or 10 000 lb. per sq. in. in tension. A subsequent compressive load of 20 000 lb. per sq. in. applied to the link will then strain the soft bars to their elastic limit in compression of 30 000 lb. per sq. in.; or a subsequent tensile load of 40 000 lb. per sq. in., applied to the link, will then strain them to their elastic limit in tension of 30 000 lb. per sq. in., and the elastic field, as before, will be 60 000 lb. per sq. in. If the compressive load, instead of being 20 000 lb. per sq. in., were enough greater (10 000 lb. per sq. in. of link greater, or 20 000 lb. per sq. in. of hard bars, which must now carry all the additional load, the soft bars being the "lazy horse") to strain the hard bars to 30 000 lb. per sq. in., all the bars, hard and soft, would become entirely free from balanced internal stresses when the link was relieved of its load.

When the hard and soft metals, instead of being segregated in separate bars, are intimately mixed in each bar, the action is more complicated, but the principles which govern are the same.

According to Thomson's theory, initial internal stresses can be eliminated and the elastic field shifted at will by over-straining.

Thomson's theory takes no account of the mobile or viscous films of metal which result from over-straining,* nor of the hardening pro-

* Subsequent to the promulgation of Thomson's theory of over-straining, he wrote a paper, in 1861, on crystallization and liquefaction as influenced by stresses tending to change of forms in crystals.

duced by their resolidification. Instead, he based it in part on the tentative proposition: "That no change in the hardness of the substance composing the material has resulted from the sliding of its particles," after it has been strained beyond its elastic limit. Mr. Prichard.

The viscous films will solidify and cement the adjacent crystalline surfaces during a rest, whether the metal is free from stress or is stressed, or even while the stress is fluctuating, provided the fluctuations are not alternations.

Moderate over-straining followed by rest or by only moderate fluctuations tends to increase the elastic limit of the metal by liquification and resolidification in harder, stronger form of the weakest films or portions; that is, the portions having the lowest elastic limits, and this tends to increase the elastic field, as illustrated in Table 10.

Alternations of stress appear to be very fatiguing by hindering the resolidification of the viscous metal and by squeezing it from between the sliding crystalline surfaces, and thus producing permanent injury and weakness in the form of cracks.

Alternations of stress may actually take place when repeated stresses seem to be all of one kind; for instance, in the case of the link of soft and hard steel bars, previously analyzed, repeated loads from zero to 40 000 lb. per sq. in. would involve repeated alternations from 30 000 lb. per sq. in. tension to 10 000 lb. per sq. in. compression.

If at any time the stress fluctuates without rest and the fluctuations exceed twice the elastic limit of what is at that time the weakest portion of the metal, over-straining and stress reversals, the conditions which make for permanent fatigue, are bound to ensue, no matter how well the elastic field is adjusted to the conditions; and even when the range of fluctuations exceeds simply the elastic limit of the said weakest portion, over-straining and stress reversals are likely to ensue if the conditions as regards balanced internal stresses are sufficiently unfavorable.

Any piece of steel manufactured by hot-rolling or hot-forging is, to a greater or less extent, strained initially by balanced internal stresses, besides which it contains microscopic flaws which cause unequal distribution of stress from external forces; hence (assuming that its condition as regards balanced internal stresses has not been adapted by previous suitable over-straining and rest treatment to the particular stress conditions to which it is to be subjected) the range between the limits within which the loading can continuously fluctuate without producing both mobile metal and alternate stresses, must in practice for such a piece be less than twice the elastic limit of its weakest constituent metal. The elastic field of such a piece is naturally normal; that is, its yield point and the imperfections in elasticity within the yield point may naturally be expected to be about the same in tension as in compression; and the range of fluctuation, which will

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not, at the start of a continuous series of stress fluctuations, produce more mobile metal than is consistent with subsequent endurance, will be greatest when the fluctuation is between equal tension and compression, less when the fluctuation is between zero and an upper limit of tension or compression, and least when the fluctuation is between two limits of the same kind of stress, one of which is the greatest possible.

Bauschinger, Unwin, Stanton, Bairstow, and others who have critically studied cycles of stress and compared the numbers of repetitions necessary to produce rupture at different stress intensities, have concluded, after plotting the curves of rupture, that these curves indicate certain limiting stress intensities, for different materials, below which rupture would not occur, no matter how often the cycles were repeated. Mr. Moore, however, does not accept the conclusions of these experimenters and analysts. Instead, he states:

"Professor Basquin, of Northwestern University, has pointed out that, for a considerable range of stress, the results of repeated stress tests of metals are well represented by an exponential equation."

The British experimenters, Eden, Rose, and Cunningham, and the American experimenters, Upton and Lewis, also pointed out such an apparent relation. Mr. Moore further states that, as a result of investigations on which he and Mr. F. B. Seely have been engaged:

"The following modification of Basquin's formula is proposed:

$$S = \frac{B}{(1 - Q)\, N^{0.125}}, \quad * \; * \; *$$

in which

S = fiber stress necessary to cause failure;
N = corresponding number of repetitions of stress (the 'life' of the piece);
Q = ratio of minimum stress applied to maximum stress ($Q = 0$ for load varying from zero to a maximum, $Q = -1.0$ for a completely reversed load);
B is an experimentally determined constant."

Mr. Moore has stated to the writer:

"This equation is proposed as a working formula, and is based on test results near the minimum for the test data available. Many individual tests, especially those made on rotating shaft specimens under bending, show higher results than are given by this formula."

According to this formula, any cycle of stress, if successively repeated, will eventually cause failure.

The Watertown Arsenal's endurance tests of shafts of open-hearth steel of various grades, as given in Table 3, by covering a much greater range of repetitions of cycles of stress than is usual in experiments of this kind, afford a good opportunity to try out Moore's formula and the opposite contentions of the other school of analysts cited. The

writer has accordingly diagrammed these tests. They all plot quite close to smooth curves, which, as far as they go, and they go far enough to be very assuring to structural engineers, support the contention that there are limits within which cycles of stress can be continuously repeated without resulting in failure (it should be remembered, however, that mechanical engineers have to do with repetitions of stress far outnumbering anything in these tests; for instance, the rotations of shafts of steam turbines in a few years amount to billions). One of these curves is shown in Fig. 14, together with the locus of Moore's formula, which, it will be observed, does not agree with the experi- Mr. Prichard.

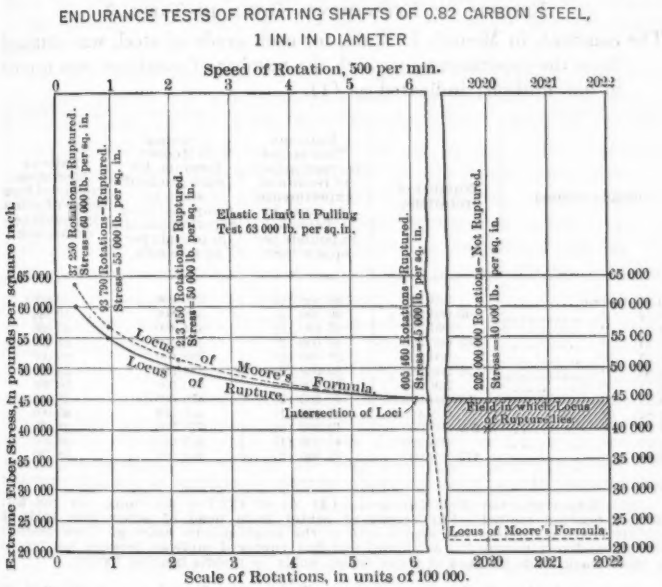

FIG. 14.

ments. Mr. Moore suggests, in a letter to the writer, "A lower value of exponent than 0.125 would give better agreement with these particular test results."

To make a further comparison of Moore's formula with the experiments in Table 3: first, the values of B were obtained for each grade of steel, by taking for each case the values of N and S from the test in which the number of rotation was closest to one million; second, by computing in each case from these values the value of S for the maximum number of rotations given in the table; third, by placing the

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values of S, thus computed from Moore's formula, in juxtaposition with the values of S corresponding to the actual results of experiments. This comparison is given in Table 11.

From Table 11 it appears that the various grades of steel sustained, without failure from fatigue, millions of alternations of stress varying for the different grades from 49 to 87% in excess of the failure stress indicated by Moore's formula. The failure of the 0.34 carbon steel at 63 667 320 rotations was evidently due to wearing and scoring at the bearings, and not to fatigue.

TABLE 11.—COMPARISON OF MOORE'S FATIGUE FORMULA WITH RESULTS OF EXPERIMENTS GIVEN IN TABLE 3.

The constant, in Moore's formula, for each grade of steel, was obtained from the experiment in which the number of rotations was nearest to one million; indicated as (1).

Grade of steel.	Number of rotations.	Extreme fiber stress corresponding to results of experiments as given in Table 3, in pounds per square inch.	Constant in Moore's formula, for each grade of steel, as computed from experiments (1), in pounds per square inch.	Extreme fiber stress computed from Moore's formula, in pounds per square inch.
0.17 carbon.....	293 510	40 000 (1)	386 800	40 030
0.17 "	100 000 000	30 000 (2)	386 800	19 340
0.34 "	166 360	45 000 (1)	404 400	45 000
0.34 "	63 667 320	40 000 (3)	404 400	21 400
0.55 "	900 720	35 000 (1)	388 600	35 000
0.55 "	75 006 090	30 000 (2)	388 600	20 140
0.73 "	238 212	50 000 (1)	470 000	50 000
0.73 "	58 400 000	40 000 (2)	470 000	25 130
0.82 "	605 460	45 000 (1)	475 300	45 000
0.82 "	262 000 000	40 000 (2)	475 300	21 770
1.09 "	433 380	40 000 (1)	405 200	40 000
1.09 "	175 280 000	35 000 (2)	405 200	18 730

(1) Ruptured; (2) Not Ruptured; (3) At 63 432 700 rotations, bar run hot; middle bearing melted. Bar scored at center from head of screw that holds up middle bearing fixture; bar also scored at the south middle bearing. New bearings put in, and test resumed. At 63 667 320 bar ruptured midway between bearings at a score mark made by head of screw which holds up middle bearing fixture.

Some of the shafts recorded in Table 3, after being subjected to many millions of alternations of stress without rupture, as indicated in that table, appear to have had their capacity for endurance enhanced by the straining they had undergone, as they were subsequently subjected to endurance tests at higher loads, which they then sustained for more rotations than the shafts which had not been subjected to previous tests; for instance, the 0.17 carbon shaft, after resisting 100 000 000 rotations at a stress of 30 000 lb. per sq. in., stood 6 470 460 rotations at 35 000 lb. per sq. in. before rupture, against 5 757 920 rotations at 35 000 lb. per sq. in. of a shaft not previously tested; the 0.73 carbon shaft, after resisting 58 400 000 rotations at

40 000 lb. per sq. in. without rupture, stood 1 000 rotations at 45 000 plus 1 000 at 50 000 plus 1 000 at 55 000 plus 74 040 at 60 000 lb. per sq. in. before rupture, against 55 390 rotations at 60 000 lb. per sq. in. of a shaft not previously tested; the 1.09 carbon steel, after resisting 175 280 000 rotations at 35 000 lb. per sq. in. without rupture, stood 1 000 rotations at 40 000 plus 1 000 at 45 000 plus 1 000 at 50 000 plus 1 000 at 55 000 plus 17 350 rotations at 60 000 lb. per sq. in. before rupture, against 17 540 rotations at 60 000 lb. per sq. in. in a shaft not previously tested; an 0.82 carbon shaft, not listed in Table 3, after resisting 62 076 660 rotations at 35 000 lb. per sq. in. without rupture, stood 140 830 rotations at 55 000 lb. per sq. in. before rupture, against 93 790 rotations at 55 000 lb. per sq. in. of a shaft not previously tested. Mr. Prichard.

The only shaft which on re-testing did not on its face plainly show increased capacity for endurance as a result of the previous endurance test under moderate stress was the 0.55 carbon one. This shaft, after resisting 75 006 000 rotations at 30 000 lb. per sq. in. without rupture, was subsequently subjected once to 100 rotations and eight times to 1 000 rotations (8 100 in all) at 60 000 lb. per sq. in., with 1 334 060 rotations at 30 000 lb. per sq. in. interspersed between times, and then ruptured after 3 280 additional rotations at 30 000 lb. per sq. in. Against this, a shaft not previously tested resisted 12 490 rotations at 60 000 lb. per sq. in. before rupture.

Mr. Moore, to whom Table 11 was submitted in advance of publication, stated in reply:

"The divergence of test results from the proposed exponential formula is noted by Basquin, by Upton and Lewis, and by Moore and Seely. The advocates of an exponential formula for repeated stress calculations claim that the test data for numbers of repetitions of stress greater than ten million are very few, that while most test data for high values of N shows results above those given by exponential formulas, some data (Wöhler) seem in fair agreement with those formulas, and that in our present state of ignorance as to results of long-time repeated stress tests (which must continue for at least ten years longer, since the European war has caused the cessation of most, if not all, of the long-time tests now in progress) some formula which assumes that the same destructive action which occurs under high stresses will continue to act with diminished intensity under low stresses is a safer guide for the designer than is a fixed endurance limit, below which destructive action is assumed to cease. For structures the stresses given by the exponential formula and those given by the older endurance limits are about the same. For high-speed machinery the exponential formulas give lower stresses than do the old endurance limits.

"Messrs. Moore and Seely have attempted (in their paper before the 1915 meeting of the American Society for Testing Materials) to modify their proposed formula for certain cases by the addition of a 'probability factor' which is given as $(1 + 0.015\ N^{0.125})$. This factor

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brings about a somewhat better agreement between the formula and results of most repeated stress tests in which N is greater than ten million. The proposed factor was purposely chosen to give conservative results."

The writer will not digress from the subject of his paper by including a discussion of the endurance of high-speed machinery parts, with their liability to accumulative vibration through synchronization of vibratory and loading periods, to centrifugal forces from unintentional and unapparent eccentricities in revolving masses, and to instability at critical or "whirling speeds", in addition to pulling, pushing, bending, and twisting loads; but, simply as a caution against snap judgments as to safety in proportioning such parts, he calls attention to the paradox that in some cases it is safer to use a shaft with a small diameter than a shaft with a large one. For instance, in a certain type of De Laval turbines the wheel with its shaft will revolve safely and smoothly at a tremendous speed (at a rate of 30 000 rev. per min. or 15 768 000 000 per year in the 5-h.p. turbines) about an axis which passes almost exactly through the actual center of the revolving mass, as distinguished from the nominal center, for the reason that the shaft being slender is flexible; whereas, if it were of much larger diameter, and correspondingly stiffer, there would be danger of intense bending stresses and of other detrimental phenomena from centrifugal force and vibration.

Professor Howe introduces the question of the effectiveness of raising the elastic limits of steel and iron by the cold-working of the material prior to putting it in service. There is no doubt that, when the cold-working strains are not too severe, the endurance to constant loads is increased. In this regard Thurston's endurance tests of wires, as given in Table 12, are pertinent. It is also very probable: first, that the material acquires a capacity for enduring stresses much higher than its primitive elastic limit before it was cold-worked, provided the range of variation in stress is kept somewhat less than the said primitive elastic limit; second, that, for occasional loads, with long intervals of rest between them, the range of variation in stress is much greater than the said primitive elastic limit. Looking at the question from a theoretical standpoint, it may also be, especially in the case of wire, that such a large proportion of the metal has been made viscous, and resolidified in much stronger form, that the elastic field will be enlarged, in some cases perhaps greatly enlarged, even for resistance to continuous cycles of stress. The question is one which can only be answered with any certainty by experience and by comparative endurance tests. Professor Howe cites some tests, but not enough to be conclusive, in his valuable and interesting article on the important question: "Are the effects of Simple Over-strain Monotropic?"* The

* *Proceedings*, Am. Soc. for Testing Materials, Vol. XIV, Part II (1914), p. 9.

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writer does not know of many others, but presents in Table 13 a few, which are pertinent, on rotating shafts turned down from plain and twisted iron rods. It is not safe to draw conclusions from a few cases, but on their face the tests in Table 13 indicate: first, that the fourth turn of the rod strained the metal too severely; second, that the cold-twisting increased the length of the endurance to a given alternating stress. Whether or not the cold-twisting permanently increased the elastic field of resistance to continuous alternations of stress does not appear.

The enhancement of the elastic limit by cold-working affords an interesting field for experimental and theoretical investigation, and such investigation would be of much practical value to the mechanical engineer, and also, as regards wire cables, to the structural engineer. In general, however, the steel in which the structural engineer is interested is hot-rolled, hot-forged, or cast.

The stresses which are developed in structures vary in intensity, and, even when the loads are frequently applied, there are long intervals between the stresses of greatest intensity and between the greatest extremes of stress. It is the difference between the frequency of the recurrence of such extremes and the recurrence of the extremes in endurance tests that the writer had in mind in making the statement, on page 1456, quoted by Mr. Stanton. High stresses only slightly within the primitive yield point, repeated or even alternated at long intervals would probably do little injury beyond a slight permanent deformation to tension members and stiff compression members, provided the metal had not been previously weakened by a number of frequent repetitions of great variations in stress.

TABLE 12.—(THURSTON'S TABLE NO. 11)* ENDURANCE OF IRON UNDER DEAD LOADS.

Per cent. maximum static load.	TIME UNDER LOAD BEFORE FRACTURE.	
	Hard, unannealed wire.	Soft, annealed wire.
95	8 days	3 minutes.
80	35 days	5 minutes.
85	Unnoted, but 1 or 2 years	261 days.
80	91 days	266 days.
75	Unbroken after several years	17 days.
70	Same results	455 days.
65	Same results	455 days (probable jar).

"Some of these wires were still unbroken in 1888, after 15 years' loading."

* *Transactions*, Am. Soc. C. E., Vol. XLI, p. 516.

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TABLE 13.—THE EFFECT OF TWISTING RODS OF "BURDEN'S BEST IRON" ON THE ENDURANCE OF ROTATING SHAFTS MADE THEREOF.

Shaft 1 in. in diameter, 37 in. between supports, loaded on 4 in. at center.

From Watertown Arsenal Report, 1903, p. 337.

Previous treatment of iron. Number of turns in 45 in.	Computed maximum fiber stress per square inch.	Number of revolutions. Speed, 500 per minute.	Remarks.
None (natural state)..	20 000	1 000 000	Not ruptured.
	25 000	1 716 380	Ruptured.
		2 716 380	Total rotations.
2.....	25 000	3 032 130	Ruptured.
3.....	25 000	12 074 960	Ruptured.
4.....	25 000	7 043 080	Ruptured.

Mr. Stanton calls attention to a phenomenon recorded in Mr. Bairstow's valuable paper* on the range of the elastic limits, as follows:

"Experiments on an axle steel with a yield point of 24.9 tons per sq. in. showed that in repeated loadings from 0 to 23.2 tons per sq. in. no sign of want of elasticity occurred until after 6 000 loadings, when a permanent extension of the order of the yield took place."

That there should be a difference in yield points in different specimens of the same steel is not remarkable. That the difference in the cases referred to was due to the manner of developing the yield seems probable. As the ultimate tensile strength of the specimen of axle steel which developed the yield point of 24.9 tons was 38.2 tons, the ratio of yield point to ultimate was 65.1%, or, for a yield point of 23.2 tons, 60.7 per cent.

It is doubtful whether a primitive elastic limit much in excess of 60% of the ultimate is a direct criterion of the maximum load which can be continuously repeated from zero to a maximum without causing failure. In some cases of high ratios the change from nearly perfect elasticity to rapid yield is so sudden and pronounced as to suggest that a slight change in conditions would have caused the metal to yield under a considerably lower stress; for instance, a piece of steel tested at the Watertown Arsenal† for comparison of testing machines, at 40 000 lb. per sq. in., or 82.9% of its ultimate tensile strength, had no set in a gauged length of 8 in., and had a modulus of elasticity of 30 000 000; but, at 40 600 lb. per sq. in., it suddenly yielded, and the load fell to 30 000 lb. per sq. in., which is 62.2% of the ultimate tensile strength of 48 240 lb. per sq. in. This specimen had the greatest drop

* *Phil. Trans.*, Royal Society, Vol. 210, p. 35.
† Report for 1904, p. 197.

at the yield point which the writer has ever seen recorded. Ordinarily, the drop is within 10% of the load at the yield point. In general, the drop in load at the yield point is greatest for soft and medium steels with high elastic limits, and it decreases with decrease in the ratio of elastic limit to ultimate and with increase in ultimate. Mr. Prichard.

For structural purposes, a pronounced yield is failure, and its careful observation in tests is of value accordingly. Low elastic limits which mark the beginning of a general yield are critical, but low elastic limits which are not followed within a few thousand pounds by a critical deformation are not very detrimental to tension members and stiff compression members, as their elasticity can be perfected by straining in service at the expense of a little permanent set. Such low elastic limits are caused, as explained by Mr. Howard, by initial internal stresses. The effect of such stresses on compression members, as also explained by Mr. Howard, is nearly akin to eccentricity. In other words, these stresses cause a deflection in compression members where, under ideal conditions, there would be none. This is one of the elements of weakness for which suitable provision is or should be made by use of some reasonable column formula.

Mr. Lynch does not present any data regarding the deformation which followed the "true elastic limit of 15 000 lb. per sq. in. or less" (which he states that plates with a tensile strength of more than 60 000 lb. per sq. in. often have), nor any other information from which to judge whether this "true elastic limit", as determined by the extensometer, was the real elastic limit of the steel, or simply the elastic limit of the piece in whatever condition as regards initial internal stresses it happened to be; which latter, owing to the initial internal stresses, may be much less, as Mr. Howard has pointed out. Eccentricity in loading in the testing machine, as Mr. Howard has also pointed out, and the non-observance of the practice of allowing "test pieces to 'rest' for a day or two before pulling", as pointed out by Mr. Speller, may likewise result in extensometer readings which are below the "real elastic limit" of the material.

Mr. McCulloch has cited eye-bars for which the micrometers indicated permanent sets under pulling loads of 5 000 lb. per sq. in.; and the writer could cite many other cases, but he does not know of any case in which the test showed that the steel of which the piece tested was composed had a real elastic limit as low as "15 000 lb. per sq. in.", nor of any case in which a piece of steel tested under repeated (not alternated) loads failed under a load of less than 35 000 lb. per sq. in., except one bar (which had unfilleted re-entrant angles where reduced in section and, therefore, was not a criterion) tested by Wöhler to failure on the 274 969th repetition of a load of 31 100 lb. per sq. in. Tests of steel and iron to failure under thousands and millions of repetitions of simple tension were made by Wöhler, as given in Table

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14, and by Bauschinger, as given in Table 15. The writer does not know of any others.

There is some difference between Mr. Lynch's claim that "Plates having a tensile strength of more than 60 000 lb. per sq. in. may, and often do, have a true elastic limit of 15 000 lb. per sq. in. or less" and Dr. Waddell's statement that "The elastic limit for medium bridge steel is 35 000 lb. per sq. in." It is hardly likely that Dr. Waddell wishes to have this general statement taken too literally and without allowance for the differences which may be caused by variation in reduction in rolling and in the temperature at which the steel is rolled, as he has previously published* the yield points indicated in tests made at the Drexel Institute from the same heat of ordinary open-hearth carbon steel, as follows:

12-IN. UNIVERSAL PLATES—HEAT NO. 33 342.

		Section, in inches.	Pounds per square inch.	Percentage of ultimate.
Average of	4 specimens,	1.510 by 0.355	38 800	60.5
" "	4 "	1.510 " 0.475	36 880	57.8
" "	4 "	1.510 " 0.755	33 420	53.8
" "	6 "	1.250 " 1.000	27 380	44.9
" "	2 "	1.250 " 0.995	25 700	43.8

The yield points were indicated by a set of 0.01 in. in 8 in., and were a little lower than indicated by the drop of the beam.

Dr. Waddell had occasion to investigate extensively the subject of the elastic limit, as appears in the valuable paper in which the Drexel Institute tests above cited were published; but there are many engineers who are not familiar with the extent of the differences in reduction in cross-section and in the temperature at which reduced, incident to the manufacture of structural steel (with its wide range of shapes and sizes), and who do not realize the effects on the elastic limit of the reduction and of the temperature at which the reduction is made. In this regard, interesting tests were made at the Watertown Arsenal of bars from an open-hearth steel ingot, forged down to different degrees and at different temperatures, as given in Table 16.

The writer fully appreciates the worth of the previous efforts of Dr. Waddell, Mr. Molitor, and other eminent engineers in combating the application to bridge design of Launhardt's and similar fatigue formulas. Largely through their efforts, such use of these formulas has declined, but, unfortunately, has not ceased. It is noteworthy, however, that in this discussion the application of such formulas to bridge design has not had a single champion.

* *Transactions*, Am. Soc. C. E., Vol. LXIII, p. 264.

TABLE 14.—WÖHLER'S TESTS OF STEEL AND IRON BARS BY REPEATED TENSIONS BETWEEN DEFINITE LIMITS. Mr. Prichard.

Material.	Form of bar.	Repeated Pulling Loads: From: Pounds per square inch.	To: (Bar broke.) Pounds per square inch.	To: (Bar did not break.) Pounds per square inch.	Ratio to ultimate. Percentage.	Number of repetitions of load.	Primitive ultimate tensile strength, in pounds per square inch.
Iron axle. Phœnix Co.	*A*	0	46 700	.....	107.6	800	
	A	0	42 800	.....	98.6	106 910	
	A	0	38 900	.....	89.6	340 853	2 specimens.
	A	0	35 000	.....	80.6	409 481	43 000
	A	0	35 000	.....	80.6	480 852	43 800
	A	0	31 100	.....	71.7	10 141 645	
	A	19 400	42 800	.....	98.6	2 373 424	Average.
	A	23 300	.....	42 800	98.6	4 000 000	43 400
	B	0	34 800	.....	80.2	37 828	
Krupp's axle steel...	*A*	0	77 800	.....	82.3	18 741	5 specimens.
	A	0	68 000	.....	72.0	46 285	99 800
	A	0	58 300	.....	61.7	170 170	101 200
	A	0	53 200	.....	56.3	123 770	101 200
	A	0	48 600	.....	51.4	473 766	85 100
	A	0	.....	46 700	49.4	13 600 000	85 100
	A	0	.....	44 700	47.3	13 200 000	
	A	48 600	.....	77 800	82.3	1 801 000	Average.
	A	38 900	.....	77 800	82.3	12 100 000	94 500
	A	34 000	.....	77 800	82.3	12 100 000	
Krupp's axle steel...	*B*	0	48 600	.....	51.4	23 546	
	B	0	44 700	.....	47.3	33 486	
	B	0	40 900	.....	43.3	65 658	
	B	0	38 900	.....	41.2	75 343	As above.
	B	0	35 000	.....	37.0	208 883	
	B	0	31 100	.....	32.9	274 970	
	B	0	.....	29 200	30.9	1 100 000	
Cast iron from locomotive cylinder.....	*A*	0	15 500	.....	.....	3 140	
	A	0	13 600	.....	.....	4 000	
	A	0	12 700	.....	.....	10 342	
	A	0	11 700	.....	.....	45 028	
	A	0	10 700	.....	.....	78 682	
	A	0	10 200	.....	.....	57 885	
	A	0	10 200	.....	.....	35 599	
	A	0	9 700	.....	.....	208 439	
	A	0	.....	9 700	.....	7 200 000	
	A	0	.....	9 700	.....	7 660 000	

The bars marked *A* had well-rounded corners at the point where the small middle part of the test bar joined the enlarged end. Those marked *B* had square corners.

Three equivalents, in pounds per square inch, have been given by different authorities for the centner zoll: namely; 97.24, 104 and 107, accordingly as they used the old German or assumed the old Prussian or the present German value for the centner. The old German centner was 100 lb. (Century Dictionary); the old Prussian pound was 467.7 grams (Johnson's and Brockhaus' Encyclopædias); the old Prussian zoll was 1.03 English inches (Webster's Dictionary), which brings 97.24 lb. per sq. in. as the equivalent of the centner zoll; this value was used in computing the above.

"According to Bauschinger (Meth. aus Mech.-Tech. Lab. in München, Heft 13, p. 36, 1886) the centner per zoll in which Wöhler gives his results is equivalent to 6.837 kilos per square cm." or 97.24 lb. per sq. in. (Encyclopædia Britannica, 9th Edition, Vol. XXII, p. 601).

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TABLE 15.—BAUSCHINGER'S TESTS OF STEEL AND IRON BY REPEATED TENSIONS BETWEEN ZERO AND SUNDRY UPPER LIMITS.

Number of Repetitions of Load are given to the nearest 10 000.

Material.	Primitive elastic limit, in pounds per square inch.	Primitive ultimate tensile strength, in pounds per square inch.	Repeated Loads. Test piece broke. Pounds per square inch.	Repeated Loads. Test piece did not break. Pounds per square inch.	Ratio of load to elastic limit. Percentage.	Ratio of load to ultimate. Percentage.	Number of repetitions of load.
Mild steel plates.....	34 900	63 800	58 700	.....	167.9	91.9	40 000
	34 900	63 800	58 700	.....	167.9	91.9	70 000
	34 900	63 800	58 700	.....	167.9	91.9	110 000
	34 900	63 800	58 700	.....	167.9	91.9	340 000
	34 900	63 800	58 700	.....	167.9	91.9	490 000
	34 900	63 800	51 500	.....	147.4	80.7	160 000
	34 900	63 800	51 500	.....	147.4	80.7	320 000
	34 900	63 800	51 500	.....	147.4	80.7	440 000
	34 900	63 800	51 500	.....	147.4	80.7	620 000
	34 900	63 800	51 500	.....	147.4	80.7	760 000
	34 900	63 800	44 100	.....	126.3	69.1	670 000
	34 900	63 800	44 100	.....	126.3	69.1	1 010 000
	34 900	63 800	35 800	.....	102.6	56.2	3 550 000
	34 900	63 800	35 800	.....	102.6	56.2	6 680 000
	34 900	63 800	35 800	.....	102.6	56.2	7 350 000
	34 900	63 800	.....	35 800	102.6	56.2	11 030 000
Mild steel boiler plates.....	39 400	59 600	47 000	.....	119.3	78.9	400 000
	39 400	59 600	47 000	.....	119.3	78.9	490 000
	39 400	59 600	47 000	.....	119.3	78.9	880 000
	39 400	59 600	41 900	.....	106.3	70.3	400 000
	39 400	59 600	41 200	.....	104.5	69.2	4 850 000
	39 400	59 600	36 700	.....	93.5	61.7	340 000
	39 400	59 600	.....	41 900	106.3	70.3	4 870 000
	39 400	59 600	.....	36 700	93.5	61.7	6 540 000
Thomas steel axle.....	39 400	89 800	58 700	.....	148.8	65.3	60 000
	39 400	89 800	58 700	.....	148.8	65.3	220 000
	39 400	89 800	58 700	.....	148.8	65.3	620 000
	99 400	89 800	44 100	.....	111.9	49.1	9 040 000
	39 400	89 800	.....	36 500	92.6	40.6	9 580 000
Thomas rail steel.....	42 600	87 400	58 700	.....	137.8	67.2	560 000
	42 600	87 400	58 700	.....	137.8	67.2	570 000
	42 600	87 400	44 100	.....	103.7	50.5	7 910 000
	42 600	87 400	36 700	.....	86.3	42.0	10 190 000
Bar iron.....	33 200	59 800	44 100	.....	133.1	73.7	670 000
	33 200	59 800	38 500	.....	116.2	64.4	9 310 000
	33 200	59 800	30 900	.....	93.2	51.7	16 480 000
	26 400	59 600	44 100	.....	166.9	74.1	240 000
	26 400	59 600	44 100	.....	166.9	74.1	640 000
	26 400	59 600	44 100	.....	166.9	74.1	840 000
	26 400	59 600	36 700	.....	138.9	61.6	7 400 000
	26 400	59 600	29 600	.....	111.9	49.6	9 110 000
Wrought-iron plate..	15 200	56 400	36 700	.....	239.7	65.1	2 280 000
	15 200	56 400	25 000	.....	191.5	52.0	5 180 000
	15 200	56 400	22 100	.....	144.0	39.1	5 190 000
	15 200	56 400	15 900	.....	103.8	28.2	5 170 000

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TABLE 16.—SUMMARY OF TENSILE TESTS OF BARS FORGED DOWN FROM AN OPEN-HEARTH STEEL INGOT IN THE DIRECTION OF ITS LENGTH, AT DIFFERENT TEMPERATURES AND WITH DIFFERENT AMOUNTS OF REDUCTION.

(From Watertown Arsenal Report, 1909, Vol. 3, p. 896.)

Diameter of specimens, 0.798 in. Sectional area, 0.50 sq. in. Gauged length, 6 in.

No. of test.	Marks.	Reduction under hammer, percentage.	Approximate finishing temperature, in degrees, Fahrenheit.	Elastic limit, in pounds per square inch.	Tensile strength, in pounds per square inch.	Ratio of elastic limit to tensile strength. Percentage.	Elongation. Percentage.	Contraction of area. Percentage.
8 658	13 *B*	6.0	1 400	64 000	108 000	59.2	14.6	23.0
8 645	13 *T*	9.2	1 400	67 000	109 800	61.0	17.0	25.2
8 657	12 *B*	16.1	1 400	70 000	111 800	62.6	16.1	29.4
8 643	11 *T*	18.9	1 400	72 000	110 000	65.5	13 1	18.6
8 644	12 *T*	32.2	1 400	70 000	111 400	62.8	16.0	29.4
8 648	3 *B*	2.3	1 600	56 000	105 000	53.3	7.1	9.2
8 634	2 *T*	7.2	1 600	53 000	108 000	49.1	9.1	11.6
8 646	1 *B*	11.5	1 600	61 090	109 800	55.6	13.1	16.2
8 647	2 *B*	27.2	1 600	60 000	110 200	54.4	12.6	16.2
8 633	1 *T*	28.4	1 600	69 000	112 600	61.3	15.5	18.6
8 637	5 *T*	2.5	1 800	50 000	104 800	47.7	7.2	9.2
8 636	4 *T*	5.6	1 800	52 000	108 000	48.1	7.6	9.2
8 650	5 *B*	8.0	1 800	51 000	108 400	47.0	9.5	11.6
8 649	4 *B*	14.7	1 800	59 000	113 000	52.2	8.6	9.2
8 635	3 *T*	28.2	1 800	62 000	112 800	54.9	11.3	14.0
8 655	10 *B*	3.6	2 000	48 000	105 800	45.4	7.8	9.2
8 656	11 *B*	6.1	2 000	49 000	108 400	45.2	12.0	11.6
8 642	10 *T*	10.3	2 000	50 000	108 400	46.1	9.8	11.6
8 654	9 *B*	13.6	2 000	52 000	108 200	48.0	12.3	14.0
8 641	9 *T*	26.5	2 000	55 000	110 400	49.1	13.3	16.2
8 640	8 *T*	31.6	2 000	57 000	111 200	51.2	14.0	25.2
8 638	6 *T*	4.3	2 200	47 000	108 400	43.4	10.5	11.6
8 653	8 *B*	14.5	2 200	51 000	110 600	46.1	12.3	14.0
8 652	7 *B*	21.8	2 200	50 000	109 800	45.6	14.0	23.0
8 639	7 *T*	27.0	2 200	51 000	111 000	45.9	15.1	27.4
8 651	6 *B*	29.6	2 200	55 000	113 000	48.7	16.0	27.4

The writer, as his previous papers* show, is in full sympathy with Mr. Molitor's efforts to secure reasonable safety for bridges when overloaded.

In conclusion, the writer emphasizes the facts: that strength which can only be acquired at the expense of undue deformation is of little value; that original faults in elasticity which can be corrected in service without undue deformation are not critical; that the real strength of any member of a structure is measured by the loads it can

* "Insufficient Provision for Counterstresses in Railroad Bridges", *Transactions*, Am. Soc. C. E., Vol. XLII, p. 547.
"The Proportioning of Steel Railway Bridge Members", *Proceedings*, Engrs. Soc. of Western Pa., Vol. XXIII, p. 344; also *Engineering News*, September 19th, 1907, p. 302.

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sustain without undue deformation under the conditions of service during the lifetime of the structure, as determined by other causes; that, in judging the real strength of structural members from tests, a knowledge of the principles of physical metallurgy is very useful; and that, in consideration of the conditions of manufacture, considerable variation in the strength of such members, even when made of steel from the same melt, should be expected.

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Paper No. 1371

A REVIEW OF THE REPORT OF CAPTAIN ANDREW TALCOTT CHIEF ENGINEER MEXICO AND PACIFIC RAILROAD EASTERN DIVISION FROM VERA CRUZ TO MEXICO EXPLORATIONS SURVEYS ESTIMATES 1858

By Emile Low, M. Am. Soc. C. E.

With Discussion by Messrs. T. K. Mathewson, H. T. Douglas, W. T. Ingram, L. Perez Castro, T. M. R. Talcott, H. M. Taylor, and Emile Low.

Synopsis.

The object of this paper is to record the achievements of an American engineer in railroad building in a foreign country under crucial conditions, to render him proper recognition, and to pay the tribute due him.

The first railroad built in Mexico was what is now called the Mexican Railway, extending from Vera Cruz to the City of Mexico, a distance of 264 miles. Barring the two short stretches at each terminus, built prior to 1858, this railroad was constructed during the period 1861-72. The construction was intermittent, having been interrupted at times by civil strife and again on account of financial difficulties.

Under the auspices of the President of the Mexican Republic, imposing ceremonies commemorating the completion of this railway were held in the City of Mexico, on January 1st, 1873, on which date it was officially opened for traffic. Between Orizaba and Boca del Monte

occurs the famous Maltrata Incline, which, as an engineering achievement, is without parallel in railroad construction. This section, together with other portions of the railroad, is an everlasting monument to the ability of Capt. Andrew Talcott, an eminent American civil engineer, who located the railroad and also constructed the major portion of the line.

Before giving a short biography of Capt. Talcott, attention is invited to extracts from the books of two well-known authors who visited Mexico and whose descriptions of this wonderful section of railroad find a responsive chord in the breast of every one who has passed over it.

"A little valley lies spread before us now, an emerald embosomed in the mountains, called *La Joya,* the Jewel, in the center of which is the station of Maltrata.

"Beyond this the track literally *climbs* the mountain, approaching it by great curves. All the way up the hills you can trace the road, its serpentine trail drawn in and out the valley and along the ridges, ever and anon doubling upon itself, but ever climbing. Beyond a water tank is a narrow bridge, ninety feet long, and spanning a chasm that ends only at the valley below, two thousand feet downward.

"Glorious are the views of Maltrata obtained as the train crawls in and out of the cuts. In the last thirteen miles we have climbed over three thousand perpendicular feet."*

"But our way lies onward towards the mountains. A wildness of landscape, unpictured before, opens to the view. Here rise weird rock forms, Nature's cathedral towers and grim façades, magnificent in solitude, and awe inspiring.

"Still onward and upward lies the way. One of the most remarkable railways in the world ascends to gain this steep zone, and serpentines among sheer descents to gain the summits of abrupt escarpments, from which (a remarkable feature of the topography of the eastern slope of Mexico) the traveler looks down as into another country and climate, upon those tropical valleys which he has left below. This is the Mexican Vera Cruz railway."†

Capt. Talcott was born in Connecticut, on April 20th, 1797. He entered the West Point Military Academy on April 9th, 1815, and was graduated and promoted in the United States Army to Brevet Second Lieutenant, Corps of Engineers, on July 24th, 1818, and to Captain. on December 22d. 1830. He resigned his commission in

* "Travels in Mexico", by F. A. Ober, 1884.
† "Mexico", by C. Reginald Enock, 1909.

the Army on September 21st, 1836. During this time, he had had charge of many engineering projects, and supervised the construction of important fortifications, among which were the defences at Hampton Roads, Va., Fort Adams, R. I., Fort Delaware, and Fort Monroe, serving on the last named as Superintending Engineer of Construction, from July 20th, 1828, to July 31st, 1834.

He was also the author of the "Talcott Method" of determining latitude, of which practical use was first made by him as Astronomer in the determination of the boundary line between the States of Ohio and Michigan, in 1832-36.

During his civil life, among many other engagements were those of Adjunct Chief Engineer, New York and Erie Railroad; Chief Engineer, Richmond and Danville Railroad, 1848-55; Chief Engineer, Eastern Division, Ohio and Mississippi Railroad, resigning from the latter to become Chief Engineer of the Mexico and Pacific Railroad, on December 1st, 1857, at a time of life when most men think seriously of retiring from active duties. After the presentation of his report in April, 1859, Capt. Talcott was otherwise engaged, but resumed his connection with this railroad early in 1862, resigning his position on February 10th, 1867, when the nearly completed work was suspended at the time of the withdrawal of the French Army from Mexico.

He died on April 22d, 1883, in Richmond, Va., being at the time the oldest graduate of the West Point Military Academy.

The paper is divided into eight parts, as follows, and, for a clearer understanding, the main reports are given verbatim, in order that there may be no perversion of facts:

1.—Introduction.
2.—Historical.
3.—Physiography.
4.—Explorations and Surveys.
5.—Report of Capt. Andrew Talcott.
6.—Summaries of Special Reports, *A, B, C, D, E,* and *F.*
7.—Maps, Profiles, Plans, and Estimates.
8.—Appendices *A, B, C, D, E,* and *F.*

It is opportune to state here that the original records used in the compilation of this review are now the property of Col. T. M. R. Talcott, of Richmond, Va., one of the two living sons of Capt. Talcott,

and a former member of the American Society of Civil Engineers, who kindly furnished the data for publication. Other data were obtained from various sources.

1.—Introduction.

In all civilized countries, before the advent of railroads, highways of more or less elaborate construction formed the means of communication. Mexico was no exception to this rule. Two highways, by divergent routes, connected Vera Cruz, the seaport of Mexico, with the City of Mexico, the capital of the country. One of these highways, known as the northern, passed to the north of the mountain, Cofre de Perote (12 500 ft. high), and the other, the southern, to the south of the mountain, Pico de Orizaba (17 873 ft. high). A straight line drawn from Vera Cruz to the City of Mexico passes about midway between these extinct volcanoes, the air-line distance being about 200 miles.

The peaks of the volcanoes are about 30 miles apart and are connected by a ridge, the lowest elevation of which above sea level is about 10 000 ft. Puebla was and is now the largest city between the coast and the capital. Both highways passed through this city, or it might be more properly said, they joined here, there being only one main highway between Puebla and the City of Mexico.

North of Puebla is the mountain of Malinche, an isolated peak rising 6 000 ft. above the surrounding plain. A branch of the northern highway passes to the north of this extinct volcano, through Huamantla, joining the lesser Puebla-City of Mexico *camino real* west of Apizaco. The main city on the northern highway is Jalapa and that on the southern, Orizaba. Still farther west are the well-known volcanoes, Popocatepetl and Ixtaccihuatl, the slopes of the latter extending some miles to the north. These mountains fixed absolutely the routes of travel, first the trails used by the native Indians, next the highways built by Cortez and his followers, and finally the location and construction of railroad lines.

The highways were built about 1800 by the consulates of Vera Cruz and Mexico. Later, the one through Jalapa was virtually abandoned, mainly on account of the larger number of important towns along the southern highway. Although there was no appreciable difference in length, it took 4 days to make the journey by diligence

from Vera Cruz to the City of Mexico *via* Jalapa, and only 3 days *via* Orizaba.

These roads, notwithstanding their dilapidation due to gradual neglect, were originally admirable specimens of solid construction, as were also the bridges on them, the Puente Nacional on the Jalapa Section deserving special mention.

2.—Historical.

On August 31st, 1837, under the administration of Gen. Anastacio Bustamante (who was President from 1837 to 1839), Don Francisco Arrillaga, a merchant of Vera Cruz, obtained an exclusive privilege for the construction of a railway from the City of Mexico to Vera Cruz, with a branch to Puebla. The proposed line had one great drawback, namely, that it only united Vera Cruz, Puebla, and the City of Mexico, leaving Cordoba and Orizaba isolated on one side, and Jalapa on the other.

The projector estimated the cost of the main line at $5 000 000 and that of the Puebla Branch, leaving at Apizaco, at $500 000. After some surveys had been made, the concession was forfeited, no construction work having been done.

A new concession was granted on May 31st, 1842, by President Don Antonio Lopez de Santa Ana, for the construction of a railroad from Vera Cruz to the Rio de San Juan, in the State of Vera Cruz, about 20 miles west. A tax formerly known as *de averia* (highway repairs) was re-established, and the proceeds were applied to railroad building and to repairs of the Jalapa Highway. During the 7 years of this grant, the company holding it constructed only 1 league of railroad and then abandoned the concession, the Government confiscating the works and prosecuting them until 1854, when a tramway had been built to La Caleta (a suburb of Vera Cruz) and the railroad was extended to the San Juan River.

On August 2d, 1855, Messrs. Mosso obtained a charter to construct a railroad from San Juan (near Vera Cruz) to Acapulco, virtually from the Atlantic to the Pacific. The resulting operations were the construction of a short stretch of railroad about 3½ miles long from the City of Mexico to the neighboring village of Guadalupe Hidalgo, which was opened for traffic on January 11th, 1857. This short line and the somewhat longer one out of Vera Cruz to Tejeria comprised

at this date the entire railway system of Mexico, some 20 miles, much of which was built under the direction of the late Robert B. Gorsuch, Hon. M. Am. Soc. C. E., later an Assistant under Capt. Talcott.

The Mosso concession was sold to Don Antonio Escandon, who, on August 31st, 1857, obtained from the Mexican Government the right to construct a railroad from Vera Cruz to the Pacific Ocean. Señor Escandon also bought from the Government the line between Vera Cruz and San Juan, near Tejeria. Previous to this date all attempts to connect Vera Cruz with the City of Mexico by rail were abortive, and it may be said that the realization of what ranks as a great engineering enterprise began at that time.

The dominant spirit in this great undertaking was Don Manuel Escandon, who placed his younger brother, Antonio, at the head of it, because he realized (as actually happened) that he himself would probably not live long enough to carry an enterprise of such magnitude through to completion. The death of Don Manuel Escandon occurred in the City of Mexico, on June 7th, 1862.

3.—Physiography.

The surface features of Mexico consist of an immense elevated plateau, with a chain of mountains on its eastern and western margins, which extend from the United States frontier southward to the Isthmus of Tehuantepec; a fringe of lowlands (*tierras calientes*) between the plateau and the coast on each side; a detached, roughly mountainous section in the southwest, which belongs to the Central American Plateau; and a low sandy plain covering the greater part of the Isthmus of Yucatan. The Peninsula of Lower California is traversed from north to south by a chain of barren mountains which cover the greater part of its surface. The slopes are precipitous on the east coast, but, on the west, they break down in hills and terraces to the Pacific. This range may be considered to be a southward continuation of the California Sierra Nevada.

The Great Plateau of Mexico is very largely of volcanic origin. Its superstructure consists of igneous rocks of all descriptions with which the original valleys between its marginal ranges have been filled by volcanic action. The remains of transverse and other ranges are to be seen in the isolated ridges and peaks which rise above the

level of the table-land, in some cases forming well-defined basins; otherwise, the surface is singularly uniform in character and level. The two noteworthy depressions in its surface, the Valley of Mexico and Bolson de Mapimi, once contained large bodies of water, of which only small lakes and marshy lagoons now remain.

The highest part of this great plateau is to be found in the States of Mexico and Puebla, where the general elevation is about 8 000 ft. Southward, the slope is broken into small basins and terraces by transverse ranges, and is comparatively abrupt. Northward, the slope is gentle, and is broken by several transverse ridges. At Ciudad Juarez (adjoining El Paso, Tex.), on the northern frontier, the elevation is 3 600 ft., which shows a slope of only 4½ ft. to the mile.

The mountain ranges which form part of the great Mexican Plateau consist of two marginal chains known as the Sierra Madre Occidental, on the west, the Sierra Madre Oriental, on the east, and a broken, weakly-defined chain of transverse ranges and ridges between the Eighteenth and Twentieth Parallels, known as the Cordillera de Anahuac. The Sierra Madre Oriental consists of a broken chain of ranges extending along the eastern margin of the plateau from the great bend in the Rio Grande southeastward to about the Nineteenth Parallel. In the north these ranges are low, and offer no great impediment to railroad building; south of Tampico, however, they are concentrated in a single lofty range. This range extends southeastward along the western frontier of the State of Vera Cruz and includes the snow-capped cone of Orizaba, or Citlaltepetl (18 209 ft.), and the Cofre de Perote, or Nanchampopetl (13 419 ft.). The eastern slopes are abrupt and difficult, and form a serious impediment to communication with the coast. Rising from the open plateau half way between this range and the City of Mexico is the isolated cone of Malinche, or Malintzin (14 636 ft.). Some 20 miles southeast of the City of Mexico are the gigantic snow-clad volcanoes of Popocatepetl (Smoking Mountain) and Ixtaccihuatl (White Woman), 17 888 and 17 343 ft., respectively.

The lowland or *tierra caliente* region, which lies between the Sierras and the coast on both sides of Mexico, consists of a sandy zone of varying width along the shore line, which is practically a tide-water plain broken by inland channels and lagoons, and a higher belt of land,

rising to an elevation of about 3 000 ft., formed in great part by the débris from the neighboring mountain slopes.

The peculiar surface formation of Mexico, a high plateau shut in by mountain barriers with a narrow lowland region between it and the coast, does not permit of the development of large river basins. The hydrography of Mexico, therefore, is of the simplest description, a number of small streams flowing from the plateau or mountain slopes eastward to the Gulf of Mexico and westward to the Pacific.

4.—Explorations and Surveys.

On December 2d, 1857, Don Antonio Escandon, in consultation with Gen. Manuel Robles, who represented the Mexican Government at Washington, engaged the services of Andrew Talcott, former Captain in the Corps of Engineers, United States Army, to make surveys for a railroad from Vera Cruz to the City of Mexico.

Immediately after his appointment Capt. Talcott organized a corps of engineers, in the City of New York, which sailed from New Orleans, La., on January 1st, 1858, and landed in Vera Cruz on January 4th. The members of this corps were: Capt. Andrew Talcott, Chief Engineer; Andrew B. Gray, Quartermaster and Commissary; J. S. Stoddart, Richard Page, and L. Van Wyck, Assistant Engineers.

First Brigade.—M. E. Lyons, Chief Engineer; E. M. Richards, R. F. Stack, T. W. Sargent, W. W. Dechert, A. C. Hall, E. McLenegan, J. Radnich, James J. Arnold, and George E. Clymer.

Second Brigade.—R. B. Gorsuch, Chief Engineer; D. S. Crosby, W. R. Eastman, A. Van Burcke, James Emmet, T. A. Emmet, H. G. Webber, L. B. Ward, O. W. Boynton, Alfred Delano, and D. M. Gorsuch.

Exploring Party.—Sidney Coolidge, Chief; T. M. R. Talcott, Charles Miller, C. A. Wolf, and George R. Talcott.

Of the foregoing, Messrs. T. M. R. Talcott (son), of Richmond, Va., and W. R. Eastman, of Albany, N. Y., are still living.

On his arrival Capt. Talcott found the country in a state of turmoil; for 3 years Mexico was a prey to civil war between two rival governments, the Republicans at Vera Cruz under Juarez, and the Reactionaries at the Capital.

With a view to an examination of the two principal routes from Vera Cruz to the table-land, one *via* Orizaba, and the other *via* Jalapa, two brigades of engineers had been organized before leaving the United States, as previously indicated.

Viewed from the ocean side, in the State of Vera Cruz, the almost perpendicular rocky walls, rising to a height of 10 000 ft. and more above ocean level, seemed to preclude the possibility of constructing a railroad from the lowlands to the plateau, but this was the problem which confronted Capt. Talcott and which he was called on to solve.

Comparative quiet having set in, the surveys of these two routes progressed favorably until the Constitutional President, Benito Juarez, who had escaped by the Pacific Coast, arrived at Vera Cruz, and military operations became more general, with the inevitable *pronunciamientos* which unsettled conditions wherever they occurred and made it difficult for the engineering parties to secure proper passports from recognized authorities to enable them to carry on their work without interruption and accomplish it at reasonable cost.

The cost of the surveys was being paid by Don Manuel Escandon, and they would not have been undertaken at this time if he had foreseen the breaking out of a revolution; but he was not willing to abandon them until it was determined that it was practicable to build a railway from Vera Cruz to the City of Mexico, and ascertain approximately what it would cost.

For these reasons, it was decided to reduce the force of engineers, and as the line *via* Orizaba was more advanced and conditions were less threatening on that route, it was deemed best to discontinue the survey *via* Jalapa until some future time, and concentrate the engineering force which was retained, in order to complete the survey of the Orizaba route as quickly as possible.

Previous to the arrival of Capt. Talcott, the Jalapa route had been examined by Don Pascual Almazan, a Mexican engineer. Subsequently, this route was examined (reconnoissance) by one of Capt. Talcott's assistants, Mr. E. M. Richards, whose report was unfavorable.

The Orizaba route was preferred, first, because it passed through towns of greater importance and a more cultivated and productive country; and second, because, notwithstanding the fact that it required great and difficult works, the line presented greater facilities than that by Jalapa, where the large number of ravines and the harder

nature of the soil would have required a much heavier outlay, without meeting the resources offered by the Orizaba route.

By the terms of the concession granted to Señor Escandon by the Mexican Government, surveys for two routes were required, one carrying the main line through Puebla, the other connecting Puebla with the main line by a branch. This was accomplished in the first case by running a line from the Maltrata Pass to Puebla, and thence to Apizaco in the Llanos de Apam, and, in the second case, by running a northerly line from the same point direct to Apizaco, the surveyed line from Puebla to Apizaco thus serving either as a part of the main line or as a branch line, as would be decided later.

On the completion of the surveys, in October, 1858, the engineers returned to the United States, and, under date of March 7th, 1859, Capt. Talcott forwarded to Señor Don Antonio Escandon his report "of the cost of constructing, maintaining, and operating a railway from Vera Cruz to the City of Mexico."

The receipt of this report was acknowledged by Señor Don Antonio Escandon from Mexico, under date of April 18th, 1859, and the report was also referred to in a letter from Tucabaya, dated May 19th, 1859, by Señor Don Manuel Escandon, a brother. In the former, Señor Escandon said:

"I consider it as a positive fatality that the unfortunate state of Mexico, with the civil war which now exists, prevents me for the moment from continuing the prosecution of a work which would most certainly become the solution of the real peace and prosperity of my country."

The report, as submitted by Capt. Talcott, is as follows.

5.—Report of Captain Andrew Talcott.

"Sr. Don Antonio Escandon,

"President of the M. & P. R. R. Co.

"Sir: On the 4th December, 1857, you engaged my services as Chief Engineer of your railway enterprise. It was at that time stipulated and agreed that the surveys between the port of Vera Cruz and the City of Mexico should be prosecuted vigorously and with a view to such a development of the topography of the country as would enable you to select the route of your proposed railway, define the same by maps, profiles, and specifications, and submit them to the Supreme Government for approval on or before the 31st of April following. This was one of the conditions set forth in the Decree of 31st of

August, 1857, granting you the exclusive privilege of connecting by a railway, the Capital of the Republic with the commercial emporiums of the Gulf and Pacific Coasts, Vera Cruz and Acapulco, or any other port, that might be deemed more eligible for the rapidly increasing commerce of the Pacific Coast.

"The time allowed was short, for the labor to be performed, *viz.*, to organize and equip a suitable Corps of Engineers, transport them to Vera Cruz, and thence with their instruments and camp equipage to the mountain passes, develop the most practicable of them, and from it to extend the surveys to Vera Cruz, and to the Capital, by lines of more than 260 miles in extent, and partly over ground exceedingly difficult and covered with the thick undergrowth, verdure, and vines of the tropics.

"A Corps of Engineers fully equipped and prepared for the service was landed at Vera Cruz on the 4th of January, but in consequence of the robbery and loss of the mail by the preceding packet from the United States, no arrangements had been made there for the transportation of the Corps; its movements were therefore so much delayed that over four weeks had elapsed before the Engineers were fairly at work surveying and exploring the mountain passes. During this interval a revolution had been in progress, and had resulted in the overthrow of the Constitutional Government at the Capital. It was the opinion of the best informed in Mexico that the struggle for power between the two contending parties would not soon be terminated by the triumph of either, and that the surveys could not be vigorously and successfully prosecuted during the then existing state of affairs. The obligation was no longer in force requiring the route to be defined and specifications filed before the 1st of May, as that had been suspended by the 26th Article of the Decree providing for such a contingency as had actually occurred. As only a small Corps could be advantageously employed for any length of time, whilst the Civil War lasted, a large reduction of the Corps was decided upon and effected, so that two-thirds left the country by the packet which sailed on the 7th of March from Vera Cruz.

"One important fact had however been ascertained, by the explorations and surveys previous to the reduction of the Corps, enough had been done to demonstrate that a railway could be constructed from Vera Cruz to the table-lands on grades and curves, susceptible of being worked by locomotive power, economically, and that the cost of such road would not be excessive. The survey had been confined to the route *via* Orizaba, but reconnoissances however had been made *via* Perote by Mr. Almazan, previous to the arrival of the Corps, and afterward verified by one of the assistants, Mr. Richards, whose reports were not favorable to the Northern route, but it had not been decided that a railway was impracticable; that the surveys were not made

with the same minuteness by the Actopan Valley, was partly from the want of time, to which may be added the civil strife which was waging in that section from March until the Corps left the Republic in October; but for the above mentioned causes this route would have been surveyed with sufficient minuteness to make a comparison of it with that *via* Orizaba. The examinations made were, however, sufficient to show that it is greatly inferior to the Orizaba route, though not minute enough to allow comparisons to be made of length, grades, curves, and cost of construction.

"It has been stated that a road by Orizaba can be economically worked by locomotive power, nevertheless it will be a road of high grades. The route surveyed reaches the table-land by a line of 111 miles, and attains at that point an elevation of 8 101 English feet, 4 000 of which must be overcome west of Orizaba, and where the summit is, in a right line, distant less than 16 miles.

"The face of the country is, however, favorable for developing and by the survey the distance exceeds 28 miles, and can be increased several miles if desirable. The question therefore arises: Is it desirable for the purpose of reducing grade to further develop the road? This is strictly a question of economy, and can be solved with much certainty. The locomotive and train must ascend the 4 000 ft., the line may be lengthened, and the cost of construction thereby increased, and an additional weight of train would be the consequence. There is a point, however, where the cost of working the road would be a minimum, this could be fixed with perfect precision, if the adhesion of the engine to the rail was constant, but it is not so, sometimes it is as low as 14% of the insistent weight, and it goes as high as 30 per cent. The grade most economical, so far as working power is concerned, changes as the adhesion changes. When the adhesion equals 30% a grade of $2\frac{9}{10}$ ft. per hundred would be worked with the least expense of power, at 20% $2\frac{3}{10}$ ft. per hundred; and 14% $1\frac{9}{10}$ ft. per hundred. These are the grades for up traffic only. Should the grade be increased to $4\frac{6}{10}$ ft. per hundred, the cost of power would only be increased $7\frac{2}{10}$%, with the adhesion of $\frac{1}{5}$th or 20 per cent. As the adhesion diminishes, the cost of working increases. (See Note *A*.)

"Thus far we have considered the expenses of but one item of railway transportation, that of power. The maintenance of way is another item generally as large on ordinary roads as the cost of power. Every mile added to lessen the grade increases the cost of keeping the road in repair, and the time required to travel over it. The greatest economy lies therefore in grades exceeding those that require the minimum power, even if the cost of construction be not thereby diminished, but under ordinary circumstances with higher grades the line would be shorter, and the reduction of cost greater in proportion, than the reduction of length. If a mountain line be 10 miles long,

with grades of 4 ft. in a hundred, it is probable that with grades of $4\frac{6}{10}$ ft. in a hundred, there will be a reduction of at least 10% in the length and more than that in the cost. The saving of the cost of this mile, and of the keeping of it in repair, will fully compensate for the additional power required for working the increased declivity. But the minimum cost of constructing, maintenance, and workings will depend on the amount of traffic, as well as the cost of power and maintenance, and from my investigations it appears that the most economical grade will not vary much from $4\frac{6}{10}$ ft. in the hundred, and this may be safely assumed as the maximum grade of the mountain road above Orizaba. The location which has been made with a ruling grade of 4 ft. in a hundred may therefore be revised, and a considerable saving thereby made, and curves of 400 ft. radii substituted for those of 350 ft., which in the experimental location were admitted on the mountain line. In all other portions the curvature is unobjectionable.

"By the terms of the grant, Vera Cruz, situated on the Gulf of Mexico in Latitude 19° 11′ 52″, was to be the eastern terminus of the railway. From this city a railway had already been constructed for about 14 miles, the title to which having been transferred to you, it was a matter of some importance to include as much of this road in your proposed line as could be advantageously used. It was found that about 9 miles could be used by the route *via* Orizaba, and an experimental location was made from this point to Cordoba, a distance of 25 leagues from Vera Cruz.

"This was as far as it was expected to extend the surveys at the time the Corps of Engineers was reduced in March, but this part of the country having become in a measure tranquil, and the labors of the Engineers uninterrupted, the experimental lines were continued up the country to the summit of Maltrata Pass and thence by Cuesta Blanca and Acacingo to Puebla, and by the Plains of Apam to Mexico. A second line was also run, branching from the first near the summit, and passing by a more northern route near Nopalucan and Huamantla, and intersecting the first line again some distance northwest of Tlaxcala. All these lines have been mapped and profiled, and made the basis of an approximate estimate of cost of the railway from Vera Cruz to the City of Mexico. It should be borne in mind that these are first experimental lines, and have not been perfected or revised into an 'Experimental Location' but still they afford very good data for an approximate estimate. It cannot be doubted that in many places ground better adapted may be found, and this is most especially the case going from Puebla to the Plains of Apam. In some places the cost may be considerably reduced, or better grades secured at the same cost. Grades have been laid down on the profiles of all the surveyed lines, and the quantities of excavation and embankment required to form a roadbed 16 ft. wide, with suitable slopes, have been carefully

calculated and entered upon the profiles, from which tables of quantities have been compiled which will be found appended hereto (marked 1).

"It appears from these surveys, that a railway *via* Puebla to the City of Mexico is $22\frac{8}{10}$ miles longer, than *via* Nopalucan, and the cost will be much greater; but Puebla is a city of too much importance to be excluded from such an enterprise, and must be served by a branch, if the more northern route is adopted. Which of the two projects should have the preference involves numerous considerations, and one very important one is the amount of direct traffic between Vera Cruz and Mexico. The greater it is, the stronger will be the objection to transporting it *via* Puebla, which not only increases the distance of carriage, but descends into the Palmar Valley 1 002 ft., to be again carried up to the northern line 824 ft.

"There are, however, many considerations to be urged in favor of the route *via* Puebla, as the trunk line, but it is not necessary to decide on the route at present, and better light may be hereafter obtained to guide your decision.

"By Puebla your road will be $289\frac{2}{10}$ miles in length, by the northern line $266\frac{4}{10}$ miles, and as a branch to Puebla will be $29\frac{6}{10}$ miles, it will require 296 miles of road, or $6\frac{8}{10}$ miles more than by Puebla.

"The cost of completing the several lines is exhibited in the following table:

"TABLE 1.—ESTIMATE OF THE COST OF COMPLETING A RAILWAY FROM VERA CRUZ *via* ORIZABA, AND PUEBLA, TO THE CITY OF MEXICO.

" Locality.	Length of line, in miles.	Roadway.	Track.	Structures.	Totals.
Vera Cruz to Cordoba	67.8	$1 625 000	$882 000	$169 000	$2 676 000
Cordoba to Orizaba	16.3	949 000	252 000	87 000	1 288 000
Orizaba to Maltrata Summit	28.1	1 392 000	420 000	18 000	1 830 000
Maltrata Summit to Puebla	60.2	681 000	903 000	74 000	1 758 000
Puebla to Junction	29.6	599 000	448 000	18 000	1 065 000
Junction to Mexico	87.1	358 000	1 232 000	219 000	1 809 000
Totals	289.1	$5 604 000	*$4 137 000	*$585 000	$10 426 000

Total cost of roadway	$10 426 000
Total cost of equipment	*1 260 000
Amount already expended on San Juan and Guadalupe Railways and on surveys	1 400 000
Total for roadway and equipment	13 086 000
No data for estimating { General management, real estate, land damages, etc.	{

* See Notes *B*, *C*, and *D* for details of these estimates.

"As the road progresses, furniture to the amount of $2 500 per mile of road in operation will be required, and more as the road lengthens and business increases. On its completion, no less than $400 000 will

be required to equip each division, of which there must be three, and in a few years it may be expected that a further quantity will be required, all of which must, however, depend on the amount of business.

"By the northern route, the cost of main line will be less by..... $1 364 000

"The Puebla Branch and structures at Puebla are estimated at..... 1 278 000

"Difference of cost against Puebla Line only.. $86 000

"Full details of the work are exhibited in the accompanying papers, to which reference is respectfully requested.

"TABLE 2.—QUANTITY OF EXCAVATION, EMBANKMENT, MASONRY AND TUNNELING ON THE SEVERAL PORTIONS OF THE ROAD FROM TEJERIA, *via* ORIZABA AND PUEBLA TO THE JUNCTION OF THE GUADALUPE RAILROAD.

" Portions of line.	Length of line, in miles.	Excavation, in cubic yards.	Embankment, in cubic yards.	Masonry, in cubic yards.	Tunneling in linear feet
Tejeria to Cordoba.....	58.4	881 036	1 033 690	50 095	840
Cordoba to Orizaba.....	16.3	657 194	438 822	40 822	700
Orizaba to Maltrata Summit.....	28.1	429 117	607 806	179 115	660
Maltrata Summit to Puebla.....	60.2	414 519	757 685	16 289	.....
Puebla to Junction.....	29.6	174 582	449 677	33 368	.....
Junction to junction with Guadalupe Railroad....	83.9	419 015	378 051	5 114	.....
Totals.....	276.5	2 975 463	3 665 731	324 803	2 200

"The country traversed by the railway will afford all the material required for its construction except iron. Timber, though not abundant throughout the line, is sufficiently so in some parts of it for the supply of all. In the estimates, suitable provision has been made for its transportation, as well as for rendering it more enduring by some of the processes usually resorted to for that purpose where timber is expensive. An interesting commentary from Hugo Fink, Esq., on the timber of Mexico is appended to this report (marked 2).

"There are some features in this line of road that require particular notice, for though for three-fourths of the distance it presents lines of easy grades and open curves, requiring few expensive bridges, cuts, or embankments, there are exceptions.

"The first of these occurs at the Jamapa River, at the village of Soledad, where a bridge is required 650 ft. in length, and the grade

line is 100 ft. above the river at its low stage. I have estimated the cost of such a structure, all of masonry, at $216 000. No difficulty will be encountered in obtaining foundations for the piers and abutments. The next bridge of any magnitude will be over the Canalita Creek, and here also a permanent structure of masonry is estimated for at the cost of $90 000. Limestone of superior quality abounds in this vicinity, it is the first we find 'in place' on the line above Vera Cruz. Inexhaustible quarries may be opened for the supply of the Vera Cruz market, for building purposes, including moles, breakwaters, forts, etc., and the quality is so good that vessels lacking return cargoes may find a profit by ballasting with it for some ports. The next expensive structure will be for crossing the Atoyac River where it breaks through the Chiquihuite Mountains. Here a deep ravine has been cut in the solid limestone rock, the great depth and precipitous sides of which present formidable obstacles to a railway. To span this chasm on the present line of location will require a bridge 300 ft. in length at an elevation of 250 ft. above the stream, but by adopting a lower line, both the great height and the length of the structure may be considerably diminished. At all events, it would seem to be a point where an iron bridge may be used to advantage, and in the estimate I have allowed $275 000 for the construction of one. (See Note *F*.)

"The crossing of the Rio Seco will require 300 ft. of bridging, at an elevation of 55 ft., which is estimated to cost $50 000. Between Cordoba and Orizaba is the deep *barranca* which is crossed by the public road near the village of Fortin, by which name it is known to many, though to others as the Barranca de Metlac. The point at which the fewest obstacles are presented to a crossing for a railway appears to be about 2¼ miles north of Fortin, where it was found to be 1 085 ft. wide at the grade line, which was 346 ft. above the stream. The rock on the west side slopes about 45°, the eastern slope being still steeper, consequently the span of the bridge can be but little reduced, except at an enormous cost for embankments. It is a fine site for a suspension bridge, and if that kind of structure was not very objectionable on railroad lines, I should recommend the construction of one at this point as the cheapest method by which this ravine can be crossed. A better, but much more expensive plan would be an 'Iron Tubular Bridge' of two or three spans, resting on piers formed of cast-iron tubes, which would be cheaper than masonry.

"No detailed plan or estimate for this viaduct has been prepared, but from such data as I have been able to collect, it may be estimated at $1 200 000. (See Note *F*.) I do not recommend its construction at an early day, for we find that, by running the line down the eastern slopes of the *barranca*, it may be crossed on a bridge 600 ft. in length, and at an elevation of 140 ft., and where it is sufficiently wide to turn

the road on a curve of 400 ft. radius in order to ascend the opposite side. The road is thus brought back to the point of the high crossing and without the admission of grades higher than 158 ft. per mile, which is not objectionable, considering that the same curvature and much higher grades are admitted above Orizaba.

"This change will reduce the cost to $400 000. After passing the *barranca* no expensive work is required, except on the 9 miles of the mountain line near Maltrata Summit and 3 miles in descending into the Acacingo Valley at Cuesta Blanca.

"In estimating the cost of work, about 50% has been added to the prices usually paid in the United States, where free labor is employed, and the same percentage, also, has been added to the cost of equipment, to cover the transportation, or the extra cost of such portions as may be manufactured in Mexico.

"A most liberal estimate has been made of the cost of track, the rails to weigh 60 lb. per yd., of a superior quality of iron, with fished joints, on sleepers, either creosoted or cyanized, to give them greater durability; this is deemed indispensable on account of the scarcity of timber in Mexico, as well as the effect of the climate upon it when not so prepared, especially in the hot and temperate portions of the country.

"The cost of railroad work in Mexico, I believe, will, after the first year or two, be very little greater than in the United States. Labor at the commencement of operations must be taken to Mexico at the expense of the parties doing the work; but when a demand for it, with constant and steady employment at fair wages, is established and generally known in Europe, and in the United States, it is but reasonable to suppose that emigration will move in that direction to the full extent that employment can be given to it; and the cost of work should then be very little, if any, greater than in the United States. Except in the hot country, a man can labor as many hours, and with as much effect, as in any other country; subsistence and clothing are not more expensive, if as much so, as in New York or Ohio. It would not, therefore, seem to be judicious to make very large contracts at the commencement of the work; otherwise all the reduction in the cost of labor would go to the benefit of the contractor, who as a first experiment would calculate on liberal prices, and insist on a large margin to meet contingencies, the most of which a slight advance will cover, after allowing for time required for traveling and cost of transportation of such labor as must be engaged in foreign countries. The instability of the Government would seem to be the greatest obstacle to cheapening labor in Mexico, but that I conclude is a risk to be borne by the Company, and not by the Contractor.

"The following is believed to be a fair estimate of the probable cost of maintaining and operating 266½ miles of railway in Mexico, with

the number of trains that may from time to time be required for its business:

"With	2	daily	trains	$365 000	per	annum.
"	4	"	"	580 000	"	"
"	6	"	"	798 000	"	"
"	8	"	"	1 015 000	"	"

"For the details of this estimate see note (marked *E*) where the number of trains for passengers and freight has been assumed to be equal, or in some cases mixed for passengers and freight.

"The data are not in my possession for estimating the present or prospective amount of trade and travel over your road.

"The foregoing estimate of the cost of constructing, maintaining, and operating it, with from 2 to 8 daily trains, each capable of conveying 200 passengers, their baggage, and the mails, in 16 hours, or 200 tons of freight in 36 hours, from Vera Cruz to the City of Mexico, will, with that of your statistical commission, afford sufficient data for investigating the financial prospects of the enterprise. From the very imperfect observations made of the amount of business, during the progress of the surveys, though a civil war was then raging, I should estimate that more than Three Million of Dollars was paid annually for the transportation, that can be better done by a railway for one-quarter of that amount.

"The maps and barometrical profiles prepared under your authority by Mr. Almazan, and handed to me a few days after the landing of the party in Mexico, were of great value, as they enabled me to place the parties on the lines of greatest promise before a reconnaissance could be made in person. To the Engineers employed in the enterprise I am under great obligations for the prompt and satisfactory execution of all the duties assigned them, whether in the field, or in the office.

"All of which is respectfully submitted,

"ANDREW TALCOTT,

"NEW YORK, MARCH 7TH, 1859." *"Chief Engineer."*

6.—SUMMARIES OF SPECIAL REPORTS, *A, B, C, D, E,* AND *F.*

In addition to his general reports, Capt. Talcott also submitted the following special reports or notes:

A.—Effective Working of Engines on Grades.—This is a study of the tractive power of locomotives to determine the most economical grades for the Maltrata Incline. It was assumed that an engine weighing 25 tons, with a tender of 20 tons, when charged with fuel and water, could take a train of 130 tons from Vera Cruz to Orizaba; thence to the Maltrata Summit an additional engine of 25 tons, constructed for that service, would be required to assist in reaching the summit.

B.—Estimate of Cost of One Mile of Track and Keeping it in Repair until Opening of Section for General Traffic.—This contains remarks on the comparative economy of best quality with inferior rails; extract from a report of the President of the Philadelphia, Wilmington and Baltimore Railroad Company, in 1858, on the subject of rails; and extracts from a report on European railways, by Colburn and Holley, relative to rails. At the time of making this report (1858-59), merchantable rails, or such as were ordinarily manufactured for the American market, could be purchased for from $33 to $41 per ton. An estimate of $70 per ton was made for rails delivered in Vera Cruz, which allowed considerable margin for changes in the market, as well as for better rails than those ordinarily made for the United States, it being assumed that good rails should last from 12 to 16 years. The experience of the Philadelphia, Wilmington and Baltimore. Philadelphia and Reading, and also of English railroads, was cited as to weight and manufacture of rails, and makes interesting reading, showing that rail troubles were not unknown at that time.

The cost of 1 mile of track for the Mexico and Pacific Railroad was assumed at $14 000, the weight of rail per yard being 60 lb., although it is not specifically stated, but is inferred from the quantity given per mile.

This report contained the following estimate of the cost of 1 mile of track and of keeping it in repair until the opening of the section for general traffic:

2 440 cu. yd. of ballast at 60 cents	$1 464
2 750 cross-ties at 85 cents	2 337
95 tons of rails, extra quality, landed at Vera Cruz, at $70	6 650
6 tons of fish-plates at $80	480
1½ tons of bolts and nuts at $120	150
8 000 lb. of spikes	400
Dressing surface of roadway and preparing for the track	150
Transporting materials for 1 mile (average)	1 250
Laying 1 mile of track	600
Extra cost of switches per mile	125
Total cost of 1 mile	$13 606

Brought forward	$13 606
For repairs of slopes during construction, and keeping track in repair	394
Total cost of 1 mile with repairs	$14 000
Cost of rails, spikes, plates, etc.	$7 805
Cost of cross-ties	2 337
Total cost of materials	$10 142
Ballast	1 464
Labor and transportation	2 394
Total cost of 1 mile	$14 000
Cost of iron for 280 miles	$2 185 400

C.—Buildings and Structures for Stations.—The sum of $585 000 was estimated for the buildings and structures required for the principal stations of the railway from Vera Cruz *via* Orizaba and Puebla to the City of Mexico, and for way stations. Water stations, engine stalls, and repair shops were included in this item. The largest proposed expenditure was at the City of Mexico, where the sum allowed was $237 000.

D.—Equipment.—It was not thought probable that more than one passenger train daily would be required for many years, except, perhaps, on the arrival of steam packets at Vera Cruz, when an extra might be needed, and that was provided for in the following estimate. It was also estimated that one freight train could take from Vera Cruz, over any grade east of Orizaba, 160 tons of gross, or 100 tons of freight, which is equivalent to 30 000 tons per annum, a quantity greater by one-half than the present imports at Vera Cruz (1858), estimated at 20 000 tons annually. It was thought probable, however, that on the opening of the entire road, from Vera Cruz to the City of Mexico, a second freight train daily would be needed. The sum of $1 260 000 was allowed for equipment, detailed as follows:

54 engines	at	$8 000 =	$432 000
36 passenger cars	"	2 500 =	90 000
18 mail and baggage cars	"	1 500 =	27 000
90 house cars	"	700 =	63 000
60 platform cars	"	500 =	30 000
Carried forward			$642 000

Brought forward			$642 000
30 gravel cars	at	400 =	12 000
30 crank cars	"	150 =	4 500
30 hand cars	"	50 =	1 500
Cost in the United States		=	$660 000
Add 50% for cost at Vera Cruz		=	330 000
			$990 000
6 turn-tables, stationary engines, etc		=	210 000
4 mountain engines		=	60 000
Total			$1 260 000

E.—Cost of Maintaining and Operating One Mile of Railroad in Mexico.—This cost was estimated to range from $365 000 per annum for two daily trains to $1 015 000 for eight trains.

F.—Iron Girder Bridges.—Letter from William Fairbairn.—In Capt. Talcott's report, mention is made of two high river crossings, necessitating bold and costly bridges. One, the crossing of the Rio Atoyac, between Paso del Macho (Mule Pass) and Cordoba, the other, the crossing of the Barranca de Metlac, between Cordoba and Orizaba, both deep chasms in which flow torrential streams during the rainy season, but which are dry the remainder of the year.

The Rio Atoyac rises in the Chiquihuite Mountains, and flowing in a southeasterly direction, joins the Rios Chiquihuite and de San Alejo, which empty into the Rio Jamapa. The proposed railroad crossing was 300 ft. long and 300 ft. high.

The Rio de Metlac rises among the fissures of the Pico de Orizaba, and, after a sinuous course, is lost in the plains adjoining the slopes of the Cacalote and Chiquihuite Mountains. It is crossed by the Vera Cruz-Mexico City highway near the Village of Fortin. This crossing was made by a series of zigzags, down one side and up the other, the opposite banks being united by a stone bridge. At the place of the crossing of the original line of survey, this ravine is more than 1 000 ft. wide and about 350 ft. deep below the adjoining plains of Cordoba on the east and those of Orizaba on the west.

At first a suspension bridge was suggested for the railroad crossing, but this design was afterward changed to an iron tubular

bridge after consultation with Mr. William Fairbairn, the well-known English engineer of the last century.

The following extract from the report of Capt. Talcott relative to the cost of tubular bridges for the two crossings mentioned is of interest in these days of cantilever structures:

From the letter of Mr. Fairbairn, * * * it appears that iron plates for a tubular bridge of 317 ft. span will cost in Liverpool £15 333 sterling, and that 1 034 ft., that is, one span of 400 ft. and two of 317 ft., would cost £57 166. One pier of masonry in the Barranca de Metlac, 346 ft. in height, would cost $225 000. One of cast-iron columns under a permanent compression of 1 ton per sq. in. would require 1 875 sq. in. of iron at the bottom, and 1 500 sq. in. at the top, equal to 2 tons per ft. of height, or 850 tons of cast-iron columns, to which should be added for the connection 150 tons, or, in round numbers, for one pier of cast iron, 1 000 tons, mostly of cast iron, which I estimate can be purchased in Liverpool, all fitted, for £9 000. For two piers, the cost of each would be about one-sixth less, or for the two £15 000. From these data I infer that my estimate of $1 200 000 for bridging the *barranca* is a very liberal one.

"MANCHESTER, NOV. 26TH, 1858.

"SIR.—In reply to your letter of 3rd I have to refer you to the annexed pen sketch of the bridges I would propose for crossing the ravines or rivers, on the line of railway between Vera Cruz and Mexico. I assume that, in crossing these valleys, it will be a single line of rails, and should this be the case the following will enable you to form a correct idea of the cost of the ironwork forming the superstructure. You will observe that I have divided the wide ravine of 950 ft. into three spaces, as being much cheaper, but I have given you the cost both ways, in order that you may draw your own conclusions as to which is the most eligible, or most expedient to be adopted.

"For the 400-ft. span we have as under:

"Tubular bridge of 400 ft. span, prepared in sections and delivered in Liverpool ready for re-construction including all the plates, bolts, rivets and expansion apparatus, the whole complete, about 1 250 tons....	£26 500
"Also, tubular bridge in three spans of 317 ft. each, collectively 950 ft. = 2 150 tons and delivered, as above..	46 000
"Total for both bridges delivered in Liverpool.....	£72 500

"To make the latter bridge of two spans, the weight will be 3 000 tons, and will cost with expansion apparatus, etc., from £65 000 to £66 000, making a difference of £20 000 in the cost of a bridge of two spans, compared with one of three spans.

"The computed strength of these tubes will be 8 tons per lin. ft., or 3 200 tons for the 400-ft. spans, and 7 600 tons for the three spans. The piers, *A A*, may be made of iron if you prefer it. For all your commands

"I am,

"Dear Sir,

"Yours sincerely,

"M. E. LYONS, ESQ., C. E." "WM. FAIRBAIRN."

Although Capt. Talcott did not recommend the immediate construction of the high Metlac Bridge, and, as an alternative, proposed a lower bridge by running the line down along the eastern side of the *barranca* to a suitable crossing and then ascending along the opposite side, the original plan of building the high bridge was adhered to for some time.

On April 26th, 1866, nearly 8 years after the report of William Fairbairn, the cornerstone was laid with appropriate ceremonies, the occasion being referred to as the "Grand Demonstration, Cornerstone Laying of Big Bridge", at which were present Señor Don Antonio Escandon, the Prefect of Orizaba, Engineers William Lloyd, Sebastian Wimmer, and others. The "demonstration" was followed by a dinner and dance.

Following this function, a pronounced earthquake occurred on May 10th, 1866, exactly 2 weeks after, which phenomenon caused the immediate abandonment of the original project, fearing for such a structure the disastrous consequences of a repetition.

The alternative line was then substituted and made a part of the location between Cordoba and Orizaba, which was run in 1866 by Gen. H. T. Douglas, an assistant of Capt. Talcott.

Under date of June 28th, 1871, Mr. William Cross Buchanan, who at that time was Chief Engineer of the Mexican Railway, as then constituted, approved this change and designed the structure crossing the *barranca,* known as the Metlac Viaduct. It consists of nine spans, each of 50 ft., and is built on a curve with a radius of 325 ft. (17° 40′), with the track at an elevation of 80 ft. above the level of the river which gives the name to the ravine. The bridge was constructed at the famous ironworks at Crumlin, England. Each pier was composed of eight cast-iron columns, of which the inner four were vertical, and the outer four had an inclination or batter of 1 in 8. These piers carried the plate girders, and these, in turn, carried the track, cross-ties, and rails.

7.—Maps, Profiles, Plans, and Estimates.

Accompanying Capt. Talcott's report was a general map showing the entire line, on a scale of 1 in. = 5 miles. There were also detailed maps of the nine divisions into which the line was divided, generally between the large cities as, Vera Cruz-Soledad; Soledad-Cordoba; Cordoba-Orizaba; these maps were drawn to a scale of 1 in. = 3 000 ft., except that for the Orizaba-Maltrata Summit Division, which was 1 in. = 1 000 ft. There was also a map of the public road from Mexico to San Martin on the larger scale.

In addition, there were eight profiles of the various divisions, and plans of proposed buildings and bridges.

There was also a detailed estimate by mile sections of the cost of graduation and masonry, summarized by divisions, the names of these being given in Capt. Talcott's report in Tables 1 and 2, under the headings, "Locality and Portions of Line."

The graduation was divided into three classes: excavation, hauled into embankment, and borrowed material. On the Tejeria-Cordoba Division, 58¼ miles, the assumed unit prices per cubic yard for excavation were, between Tejeria and Purga, about 10 miles, 25 cents; between Purga and Atoyac, along the foot-hills of the Chiquihuite Mountains, about 31 miles, 50 cents; between Atoyac and east of Paraje Nuevo, about 8 miles, in the Chiquihuite Mountains, $1.50; the material at the latter place being limestone rock. On the remaining 8 miles, the excavation price was generally 30 cents, although there was some at 50 cents and some at $1.50. For "hauled into embankment" (extra haul), the assumed price ranged from 3 to 27 cents; and for "borrowed material", 25 and 30 cents. Bridge masonry was estimated at $10, ordinary masonry at $8, and dry masonry at $4. There were two tunnels on this Division, respectively, 460 and 380 ft. long, a total of 840 ft., which were estimated at $75 per lin. ft. The cost per mile ranged from $926.80 for Section No. 8 to $78 360.58 for Section No. 57, where no tunneling or high viaducts occurred. The Atoyac Bridge Section, No. 45, was estimated to cost $304 268.15, of which $275 000 was for the bridge. About one-half the estimated cost of this Division, more than $832 000, was concentrated at four points, the crossings of the Jamapa, Chiquihuite, Atoyac, and Seco Rivers, making the average cost of 54 miles about $13 000 per mile, exclusive of track.

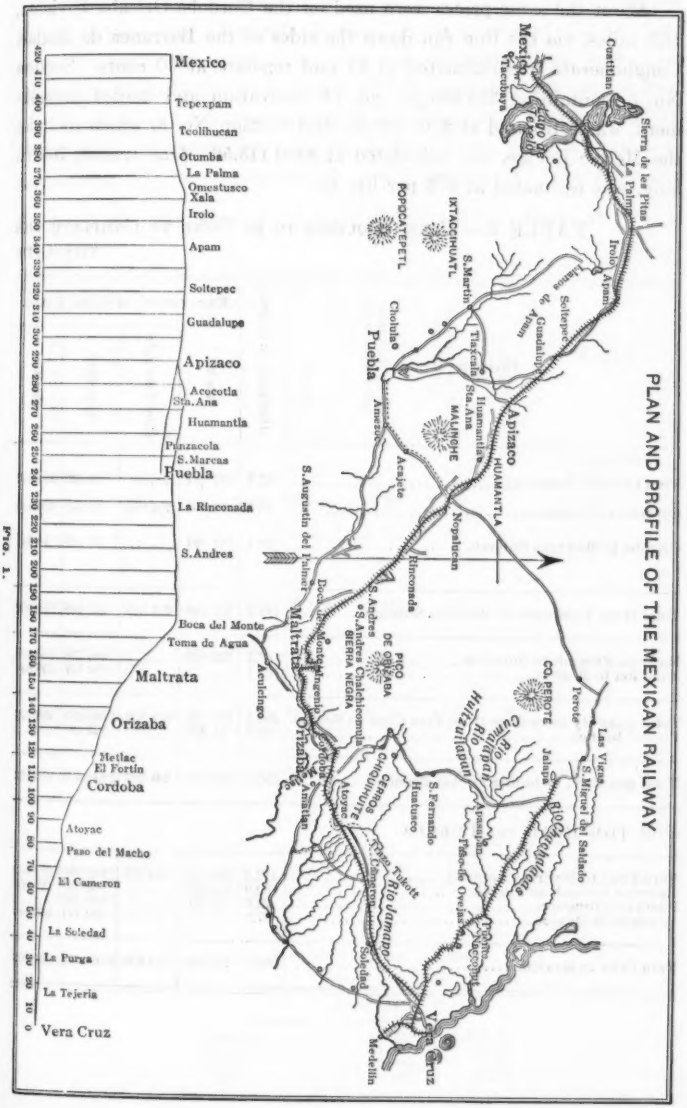

PLAN AND PROFILE OF THE MEXICAN RAILWAY
Mexico
Tepexpam
Teolihucan
Otumba
La Palma
Omestusco
Xala
Irolo
Apam
Soltepec
Guadalupe
Apizaco
Acocotla
Sta. Ana
Huamantla
Panzacola
S. Marcas
Puebla
La Rinconada
S. Andres
Boca del Monte
Toma de Agua
Maltrata
Orizaba
Metlac
El Fortin
Cordoba
Atoyac
Paso del Macho
El Cameron
La Soledad
La Purga
La Tejeria
Vera Cruz
Mexico
Tacubaya
Cuajimalpa
Cuatitlan
Sierra de las Pitas
La Palma
Irolo
Apam
Soltepec
Guadalupe
Apizaco
Tlaxcala
S. Martin
IXTACCIHUATL
POPOCATEPETL
Cholula
Puebla
Sta. Ana
Huamantla
HUAMANTLA
MALINCHE
Ameca
Acajete
Nopaluca
S. Augustin del Palmar
Rinconada
S. Andres
S. Andres Chalchicomula
SIERRA NEGRA
PICO DE ORIZABA
Boca del Monte
Maltrata
Acultzingo
Orizaba
Amatlan
Cerro Gordo
Atoyac
Cumbres
Rio Blanco
Rio Papaloapan
Rio Jamapa
Rio Tuxtepec
Rio Coatzacoalcos
S. Fernando
Huatusco
Jalapa
Perote
CO. PERROTE
Las Vigas
S. Miguel del Soldado
Paso de Ovejas
Soledad
Cruz
Medellin
Vera Cruz

FIG. 1.

About the same prices were used on the Cordoba-Orizaba Division, 16.3 miles, *via* the line run down the sides of the Barranca de Metlac. Conglomerate was estimated at $1 and tepetate at 50 cents. Section No. 5, containing 315 435 cu. yd. of excavation and hauled embankment, was estimated at $200 120.83, and Section No. 8, which includes the Metlac Bridge, was calculated at $400 113.56. One tunnel, 700 ft. long, was estimated at $75 per lin. ft.

TABLE 3.—Work Required to be Done to Complete the the City

Division.	Distance, in miles.	Excavation, in Cubic Yards.			
		Rock.	Conglomerate.	Tepetate.	Earth.
Vera Cruz to Cordoba	67.8	137 962	.....	508 632	234 442
Cordoba to Orizaba	16.3	61 695	204 809	64 571	326 119
Orizaba to Maltrata Summit	28.1	341 591	.....	73 185	14 341
Total from Vera Cruz to Maltrata Summit	112.2	541 248	204 809	646 388	574 902
Maltrata Summit to Junction	67.1	122 083	.....	260 732	146 976
Junction to Mexico	87.1	.....	.....	182 171	236 844
Total quantity to be done from Vera Cruz to Mexico.	266.4	663 331	204 809	1 089 291	958 722
Puebla Branch	29.6	51 400	.....	123 181	.....
Total quantity, including Puebla Branch	296.0	714 731	204 809	1 212 472	958 722
For the Road *via* Puebla.					
Vera Cruz to Maltrata Summit	112.2	541 248	204 809	646 388	574 902
Maltrata Summit to Puebla	60.2	165 400	.....	139 163	109 956
Puebla to Junction	29.6	51 400	.....	123 181	.....
Junction to Mexico	87.1	.....	.....	182 171	236 844
Vera Cruz to Mexico	289.1	758 048	204 809	1 090 903	921 702

On the Maltrata Pass-Orizaba Division, 27 miles, which includes the world-renowned "Maltrata Incline", with grades of 212 ft. (and more) per mile, the excavation price was generally $1.25, with some material at 25 and some at 45 cents. Hauled embankment ranged from 3 to 16 cents, and borrowed material was 30 cents. Masonry was $8, $12, and $13, and retaining wall was $2.50. There were three tunnels, aggregating 660 lin. ft., estimated at $75 per lin. ft. The estimated

ROADBED FOR THE LINE OF RAILWAY FROM VERA CRUZ TO OF MEXICO.

EMBANKMENT, IN CUBIC YARDS.		MASONRY, IN CUBIC YARDS.			Retaining wall, in cubic yards.	Tunneling, in linear feet.	Iron girders.	Remarks.
From excavation.	Borrowed.	Dry.	Culvert.	Bridge.				
644 451	389 239	.....	15 445	34 650	.....	840	.....	Two tunnels, 1-460 ft. 1-380 "
323 475	115 347	.....	7 328	28 208	5 286	700	.....	One tunnel.
164 432	443 374	.....	.....	30 021	149 094	660	40	Three tunnels, 1-190 ft. 1-110 " 1-360 "
1 132 358	947 960	.....	22 773	92 879	154 380	2 200	40	
234 708	198 294	1 485	5 982	.....	.....	.....	.....	
202 182	175 869	736	4 378	.....	.....	.....	.....	
1 569 248	1 322 123	2 221	33 133	92 879	154 380	2 200	40	
129 278	320 399	945	32 423	.....	.....	.....	.....	
1 698 526	1 642 522	3 176	65 556	92 879	154 380	2 200	40	
1 132 358	947 960	.....	22 773	92 879	154 380	2 200	40	
296 989	460 696	3 416	12 873	.....	.....	.....	.....	
129 278	320 399	945	32 423	.....	.....	.....	.....	
202 182	175 869	736	4 378	.....	.....	.....	.....	
1 760 807	1 904 924	5 097	72 447	92 879	154 380	2 200	40	

BRIDGES.	Cubic yards of masonry.	
Jamapa.....	16 000	at $12.50
Canalita.....	10 000	" 10.00
Seco.....	7 000	" 10.00
Alejo.....	1 650	" 10.00

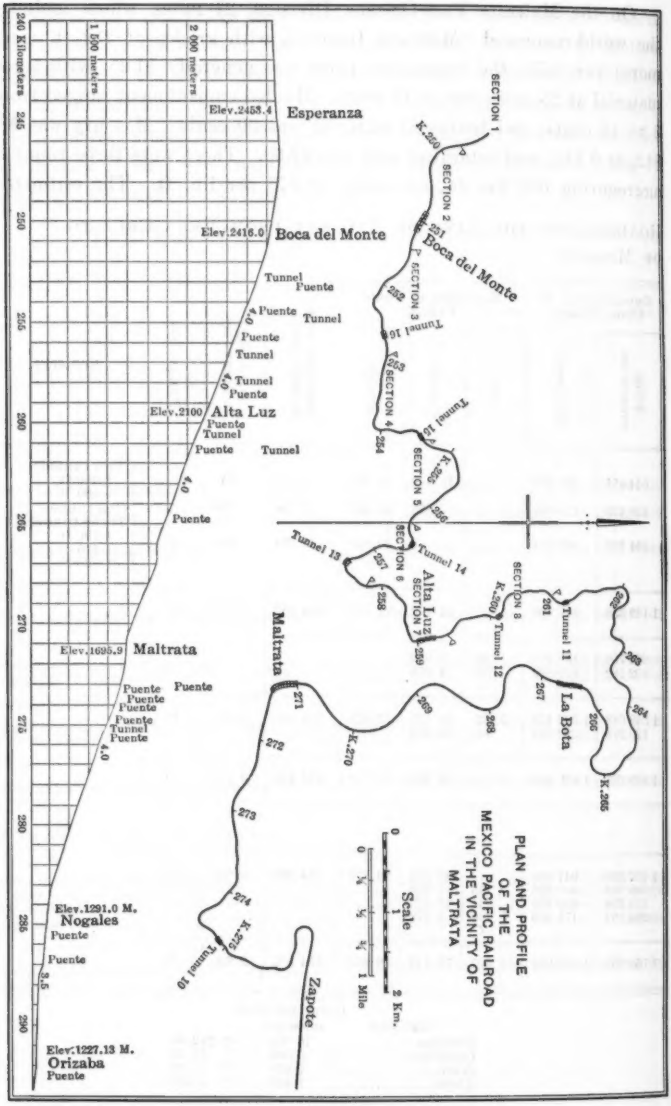

PLAN AND PROFILE
OF THE
MEXICO PACIFIC RAILROAD
IN THE VICINITY OF
MALTRATA
Scale
0 ¼ ½ ¾ 1 Mile
0 2 Km.
240 Kilometers
245
250
255
260
265
270
275
280
285
290
1 500 meters
2 000 meters
Elev.2453.4 Esperanza
Elev.2416.0 Boca del Monte
Elev.2100 Alta Luz
Elev.1695.9 Maltrata
Elev.1291.0 M.
Nogales
Elev.1227.13 M.
Orizaba
Puente
Tunnel
SECTION 1
SECTION 2
SECTION 3
SECTION 4
SECTION 5
SECTION 6
SECTION 7
SECTION 8
K.260
K.265
K.270
K.275
K.280
K.285
Tunnel 10
Tunnel 11
Tunnel 12
Tunnel 13
Tunnel 14
Tunnel 15
Tunnel 16
Boca del Monte
Alta Luz
Maltrata
La Bota
Zapotc
4.0
3.5

FIG. 2.

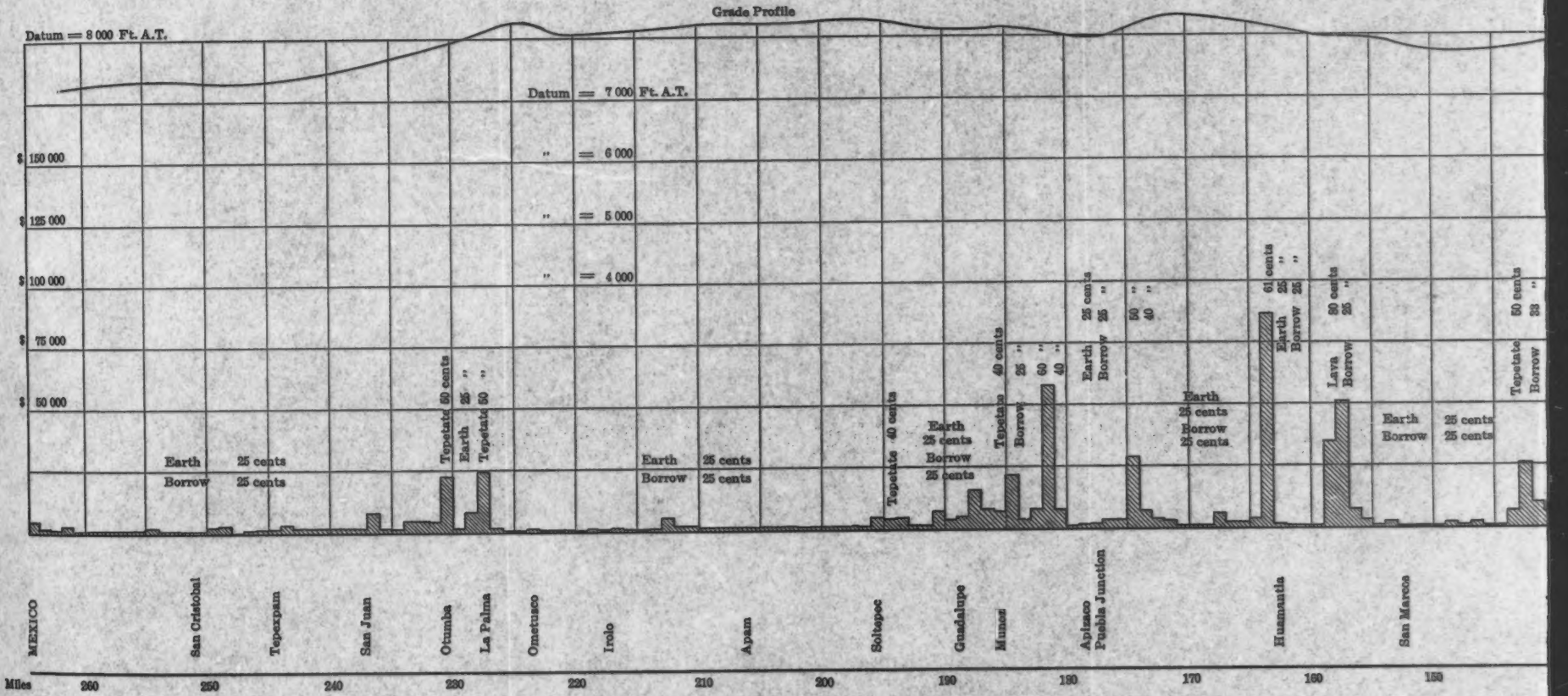
JUNCTION-APIZACO-CITY OF MEXICO
87.1 MILES

182 171	cu.yd.	Tepetate
236 544	"	Earth
175 969	"	Borrow
736	"	Dry Masonry
4 978	"	Culvert Masonry

$ 244 253	Grading and Masonry
113 797	Drainage, Engineering, etc
219 000	Track
1 222 000	Structures
440 000	Equipment
Total $ 2 249 050	
300 000	Guadalupe Road
$ 2 549 050	

MALTRATA SUMMIT-JUNC
67.1 MILE

122 083	cu.yd.	Rock
200 732	"	Tepetate
146 976	"	Earth
198 290	"	Borrow
1 485	"	Dry Masonry
5 982	"	Culvert Masonry



MALTRATA SUMMIT-JUNCTION, APIZACO
67.1 MILES

122 033	cu.yd.	Rock	$ 455 597	Grading
200 732	"	Tepetate	75 084	Drainage
146 976	"	Eearth	1 008 000	Track
198 290	"	Borrow	36 000	Structures
1 485	"	Dry Masonry	204 300	Equipment
5 982	"	Culvert Masonry	Total $1 778 981	

ORIZABA-MALTRATA SUMMIT
28.1 MILES

341 591	cu.yd.	Rock	$1 351 825	Grading
73 185	"	Tepetate	40 000	Drainage
14 341	"	Earth	420 000	Track
443 374	"	Borrow	18 000	Structures
80 021	"	Bridge Masonry	130 500	Equipment
129 120	"	Retaining Wall	Total $1 960 325	
660	lin.ft.	Tunnel		
40	"	Iron Girder		

CORDOBA
16.3 M

61 695	cu.yd.
204 809	"
64 571	"
326 119	"
115 347	"
7 323	"
28 206	"
5 286	"
700	lin.ft.

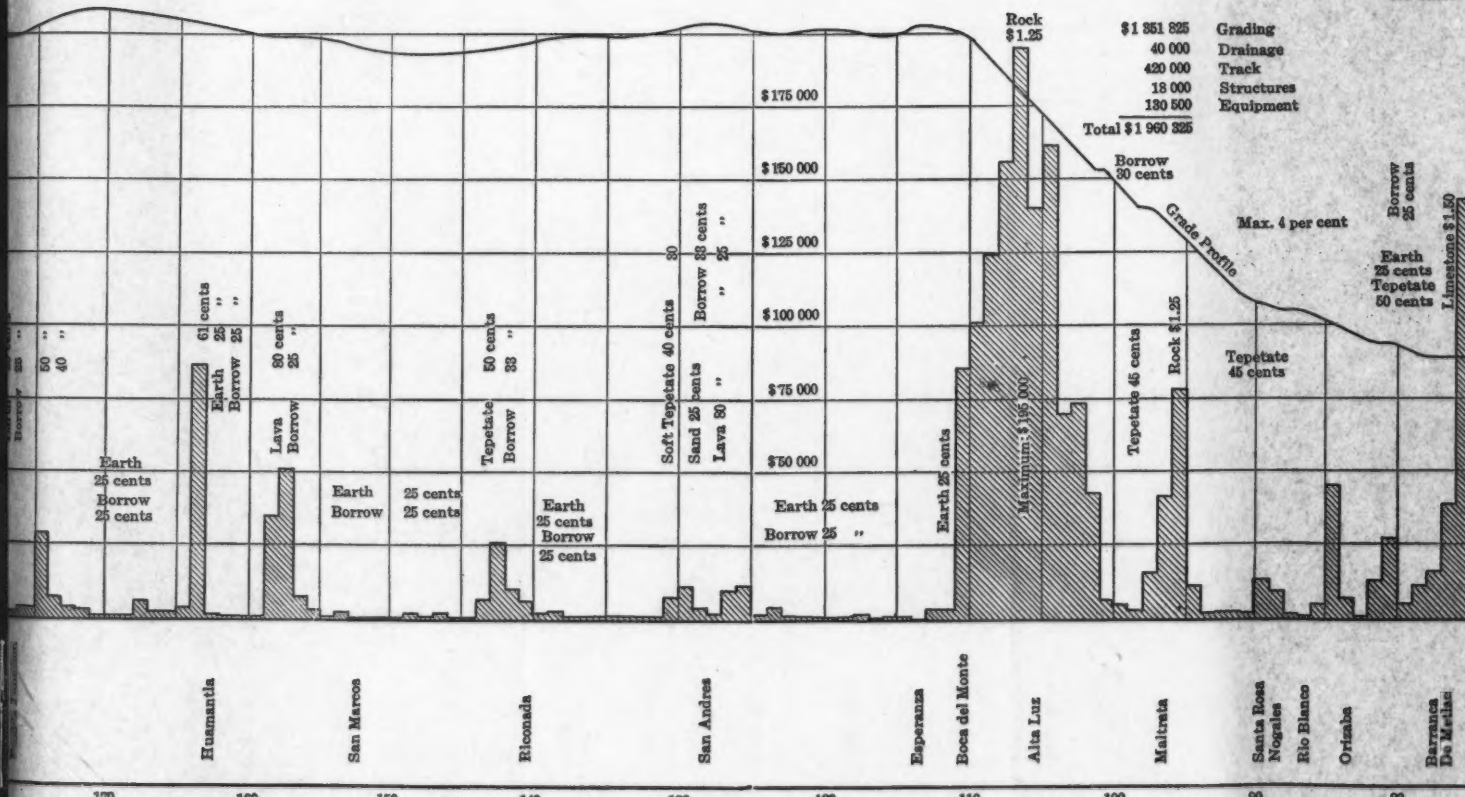


PLATE LVII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXX, No. 1371.
LOW ON
MEXICO AND PACIFIC RAILROAD.

RDOBA-ORIZABA
16.3 MILES

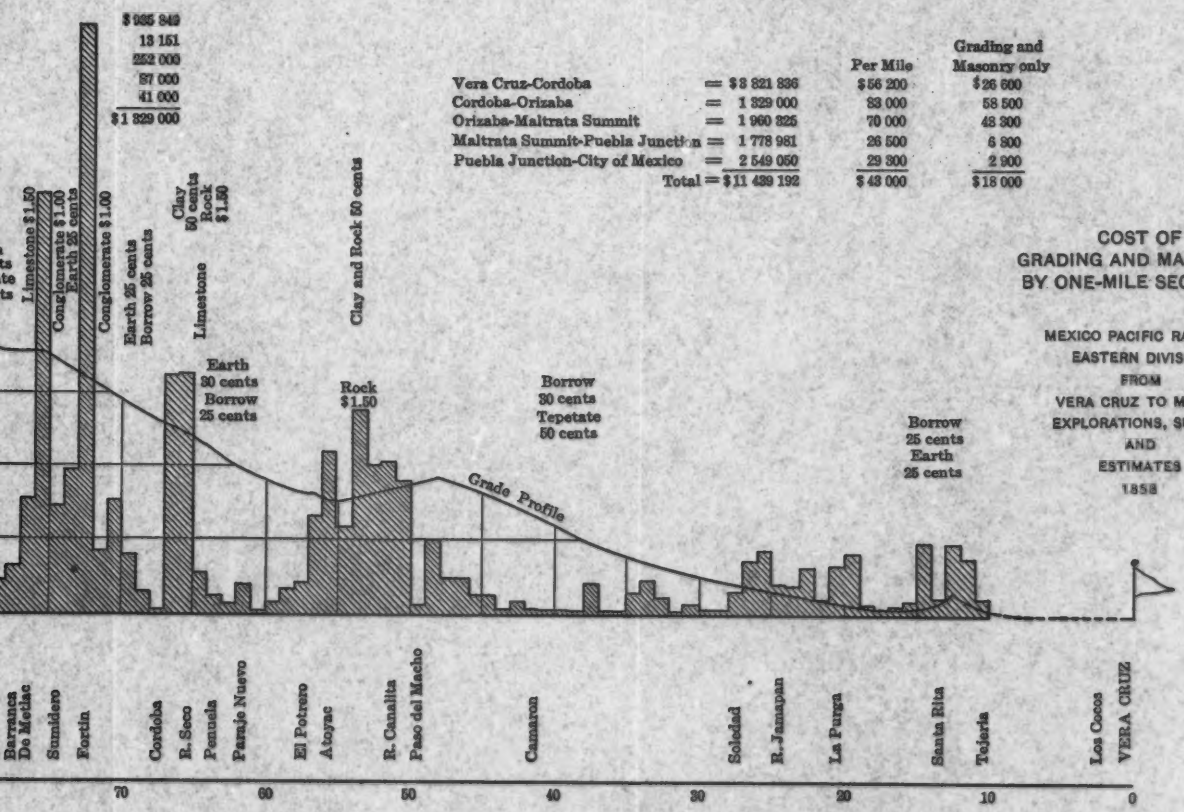
cu.yd.	Rock	
"	Conglomerate	
"	Tepetate	
"	Earth	
"	Borrow	
"	Culvert Masonry	
"	Bridge "	
"	Retaining Wall	
lin.ft.	Tunnel	

	$ 935 949
	13 151
	252 000
	87 000
	41 000
	$1 329 000

VERA CRUZ-CORDOBA
67.3 MILES

137 962	cu.yd.	Rock	$1 543 235	Grading
508 632	"	Tepetate	81 976	Drainage etc.
234 442	"	Earth	854 000	Track
389 229	"	Borrow	27 000	Structures
15 445	"	Culvert Masonry	145 625	Equipment
34 650	"	Bridge "	170 000	Mole, etc.
840	lin.ft.	Tunnel	Total $2 821 836	
			1 000 000	San Juan R.R.
			$3 821 836	

		Per Mile	Grading and Masonry only
Vera Cruz-Cordoba	= $3 821 836	$56 200	$26 600
Cordoba-Orizaba	= 1 329 000	83 000	58 500
Orizaba-Maltrata Summit	= 1 960 825	70 000	48 300
Maltrata Summit-Puebla Junction	= 1 778 981	26 500	6 800
Puebla Junction-City of Mexico	= 2 549 050	29 300	2 900
	Total = $11 439 192	$43 000	$18 000



COST OF GRADING AND MASONRY BY ONE-MILE SECTIONS

MEXICO PACIFIC RAILROAD
EASTERN DIVISION
FROM
VERA CRUZ TO MEXICO
EXPLORATIONS, SURVEYS
AND
ESTIMATES
1858

cost of Section No. 7 was $194 832.85. A large quantity of retaining wall was estimated on this Division, 129 120 cu. yd.

The next division extends from the Maltrata Summit to the junction with the Puebla Branch near Molino San Diego, 68½ miles. The excavation and borrow prices were generally 25 cents each, although there was some excavation at 50, 61, and 80 cents, the latter being lava. There was also a small quantity of borrow at 33 cents. The estimate for Section No. 38 was only $183.85, with many less than $1 000. That for Section 51 was $86 136.02. For the first 44 miles the average cost was $3 300, and for the entire 68 miles, less than $7 000 per mile.

On the direct line from the Maltrata Summit to Puebla, 61¼ miles, the excavation prices ranged somewhat higher, 25, 30, 50, 60 cents, and $1.25, the hauled embankment, from 2 to 18 cents, and the borrowed material from 25 to 50 cents. Generally, the cost per mile was low, although it was high on two sections; on No. 9 the estimated cost was $82 642.00, and on No. 10, it was $145 615.91. The average cost per mile was $10 000.

On the Puebla Branch, 29 miles, higher unit prices were assumed, uniformly 40 cents for excavation, except on Sections Nos. 27 and 28, where 80 cents was taken for tepetate. Hauled embankment was from 2 to 13 cents, and borrowed material, 30 cents.

On the division from the junction of the Puebla Branch to Mexico, called the Western Division, 83 miles, the ruling price for excavation and borrowed material was 25 cents. The average estimated cost per mile was less than $3 000.

Grades.—With the exception of the descending grades between Paso Tablo and Atoyac and the crossing of the Barranca de Metlac between Cordoba and Orizaba, there is a continuous ascent from Vera Cruz to Boca del Monte, a rise of 8 101 ft. in 111 miles. Between Vera Cruz and Paso Tablo, the range is from level to 110 ft. per mile. In constructing the railroad, a change was made, passing through Paso del Macho. In the vicinity of Cordoba and Orizaba, grades of 132 ft. per mile were proposed, and on the Maltrata Incline, 211 ft. per mile, or 4 per cent. Between Boca del Monte and the City of Mexico the grades are undulating, adapted to the configuration of the surface, the maximum used, in isolated cases, being 89 ft. per mile, but generally, of course, much lighter.

APPENDIX A

Extracts from Report of Mr. M. E. Lyons,* Principal Assistant Engineer.

"Shortly after the arrival of the engineer corps in Vera Cruz, they were instructed to proceed to Tejeria and encamp in the most eligible place in the neighborhood, until such time as arrangements could be made for transporting them to the interior, for, owing to the loss of the mails by the previous steamer, Mr. Escandon had not been duly informed of the departure of the engineers from the States; and as no means of transportation had been provided previous to their arrival, there was a consequent delay of nearly 3 weeks.

"This time was not wholly lost, however, as the party had to be drilled into camp life, surveying instruments had to be tested and adjusted, and the corps more perfectly organized.

"The threatened civil war, which afterward unhappily broke out, affected disastrously all the commercial interests of the Republic. The stories which reached us of the attacks made on travelers, the assaults committed, and robberies perpetrated on the highway in midday, made the party exceedingly wary, and induced us to form a semi-military organization, for our mutual protection, which we upheld ever afterward during our stay in the Republic.

"Sentries were mounted and relieved every 2 hours, during the night, and this was continued without intermission at each and every encampment, and to the vigilance exercised at all times the party no doubt owe their entire exemption from harm.

"The problem proposed for our solution is to find the most direct and practicable line for a railroad between the commercial and the political capitals of the Republic. Mr. Almazan furnishes a valuable map and barometric profiles which he had prepared of the Atlantic slopes. Between the mountains of Orizaba and Perote, the chain of the Cordilleras appears to be very high, and it is therefore deemed useless to make a reconnaissance of the depressions in the chain between these points. Northward of Perote and southward of the Peak of Orizaba the depressions seem to be deeper and more accessible. Two parties are detailed for the duty of examining these passes, and reporting on their practicability for railway purposes: Mr. Richards in charge of the party sent to examine the passes north of Perote, and Mr. Stoddart to examine those south of Orizaba.

"The route in the vicinity of the national road between Vera Cruz and Jalapa presents very unfavorable ground for a railroad. From San Juan, the western terminus of the existing railroad to Paso de Ovejas, passing through Zopilote, Tierra Colorada, and Tolome, it is hilly and unfavorable. The stream and valley of the Paso de Ovejas would be difficult and expensive. The country between Paso de Ovejas and Puente Nacional is thickly covered with underbrush of a tropical growth.

"The *barranca* at the Puente Nacional is a serious obstacle to a railroad, being about 1 500 ft. wide, and 250 ft. deep. For about 5 miles west of the Puente Nacional the ground is represented to be

* Mr. Lyons, formerly of Reading, Pa., was Capt. Talcott's assistant for many years.

favorable, and then becomes broken and unfavorable. At Plan del Rio we encounter another deep *barranca,* but not quite so wide as that of the Puente Nacional. At Rancho Cerro Gordo it is represented to be almost an impossible ground in a commercial sense for a railroad.

"A little west of this place, and near Mirador to Encierro, there exists a wide and deep valley descending to Dos Rios with a slope of some 240 ft. per mile, and to a depth of 500 ft. From this point to Jalapa is broken, but there is no unsurmountable obstacle in the way. From Jalapa to the summit of the Cordilleras at Las Vigas is a continuous mountain side, broken in places by ravines and water-courses, descending the mountain slopes and intersecting the route nearly at right angles.

"I cannot do better than give an extract from the report of the Engineer charged with the duty of exploring these mountain passes:

"'It [the route] certainly impressed me very unfavorably from an engineering point of view. The route from Las Vigas, round by the base of Perote [Mountain] toward Jalapa, seems—as far as I could judge from a very slight examination, which was all I could give—the most favorable; all cuttings, however, will be through rock of a volcanic nature. It comes to the surface in all directions; in many cases the débris would make very fine ballast, requiring little or no breaking. Possibly a line might be had from Las Vigas by Perote Mountain to Jalapa, with a general grade of 170 ft. to the mile, but it would be both crooked and very expensive. There seemed to be a lower gap to the north of Las Vigas. I went to examine it at one point, and Mr. Stack at another. The fog came on very thick before the exploration could be completed, but, from what I could see, it appeared about the roughest country I ever saw. The ravines were from 500 to 800 ft. deep, and the sides so steep that a man could with difficulty stand on them. This lower summit is at the head of the Canada de Actopan; up the banks of this river it has been stated that a route could be obtained; it might, possibly, but the Rio de Actopan falls into the sea considerably north of Vera Cruz, and there might be considerable difficulty in raising it out of the valley to get to Vera Cruz, except the river was practicable to the sea, which would, of course, lengthen the line considerably.'

"'From what I afterwards saw of river routes, I do not think them well calculated for railroad lines. Even if a line could be had up the Actopan River there would be great difficulty at and near the gap in question. It would require a considerable deal of exploration to pronounce on the capabilities of the region, and time did not allow me to examine it, as it ought to have been, but my general idea is that it is the worst country I ever saw to try to get a line of railway through.'

"'From Las Vigas the party returned to Jalapa, and spent some time in looking around there. It was supposed that a line might be had down a river that flows beneath the Puente Nacional, but the same river was crossed by the party two days afterward on the route from Jalapa to Cordoba. It is most likely impracticable to follow this river, if the whole of it be like the place where the party crossed it. It flows at the bottom of a *barranca* from 800 to 1 000 ft. deep. The sides often sheer cliffs from 100 to 300 ft. deep and upward. There

is no way out of the *barranca* except by a steep zigzag mule path. The rest of the country from this *barranca* to San Juan Coscomatepec is more or less broken and rugged. From San Juan Coscomatepec to Cordoba our path lay over more even ground, and formed almost a regular descending plane, crossed occasionally by small streams till we reached Cordoba.'"

Mr. Almazan, the Mexican engineer, who explored this section of country north of the Cofre de Perote, states his views in reference to the Jalapa route in his report, Appendix *B*, to which the reader's attention is directed.

ADDITIONAL EXTRACTS FROM MR. LYONS' REPORT.

"Previous to making any detailed surveys, it became necessary to determine the most practicable pass in the mountains south of the Peak of Orizaba. There were three passes which might be approached from the valleys of Ingenio, Encinal, and Maltrata. The first is the Pass of San Antonio.

"*Pass of San Antonio.*—This is the nearest to the Peak of Orizaba. The summit reached the altitude of 9 200 ft. above the sea, being more than 1 000 ft. higher than the table-land beyond. This additional ascent, over what was necessary to attain to the table-land, was a fatal objection to the adoption of this pass.

"The second one examined was the next one immediately south of the San Antonio Pass and known as the Maltrata Pass.

"*Maltrata Pass.*—The altitude of this pass was found, by barometrical measurement, to be 8 110 ft. above the sea, being at the level of the table-lands, and involving, therefore, no descent to reach them, so that when the line reached this altitude the difficulties of the Cordilleras might be said to have been fully overcome.

"*Aguila Pass.*—This was the next pass examined, its altitude being nearly the same as that of Maltrata, with this difference, however, that when the difficulty of encountering this elevation was overcome, a descent into a valley west of the summit has to be made, and then an ascent to a second summit followed by a second descent to the table-lands.

"*The Aculzingo Summit.*—This summit was also examined. The summit of the first *cumbres* was lower than any of the previous passes examined, being only 7 580 ft. above the sea; but from this summit, a descent had to be made to the valley of the Puente Colorado, then an ascent to a second summit, nearly as elevated as the Maltrata Pass, then a descent to the Canada de Ystapa, followed by another ascent to the summit beyond, from which another descent was necessary to reach the valley of San Agustin de Palmar. It became very evident that the farther south our examinations extended, the more broken was the country encountered, and that we had not only one summit to overcome but several. The Maltrata Pass was the only summit examined where, the difficulties of the first ascent being overcome, no others presented themselves, and the line ran directly on the plains.

"On further examination of this Maltrata route, we exceeded our expectations by finding gradually ascending valleys, over which the road could be carried to a considerable elevation before encountering

the obstacles of a mountain line, but this portion of the line will be more minutely described hereafter.

"Having thus accomplished what was deemed the most difficult and interesting portion of our duties, our attention was turned to discovering the most feasible line from Tejeria (the point decided on for diverging from the San Juan Railroad) to the Maltrata Pass. A great obstacle stood in our way. The Chiquihuite Mountains, which lie nearly midway between the crest of the Cordillera and the sea, had to be turned either to the north or south, or entered through the gap cut by the Rio Atoyac. Mr. Almazan had previously examined the route north of the Chiquihuite Mountains, and subsequent examinations fully corroborated the views expressed by him in his report.

"A route to the south of the Chiquihuite range had also been examined. This would cross the Jamapa River at the village of that name, the Atoyac River at Cotastla, and the Obispo River at Paso Obispo. This route was favorable, with the exception of the portion of the line between the eastern bank of the Atoyac and the Village of Los Negritos, where the ground was extremely broken and unfavorable. Another route was suggested; this would diverge from the San Juan Railroad at a point 4 miles from Vera Cruz, and, crossing the river a short distance below the confluence of the Atoyac and Jamapa, thence occupying the divide between the Rio Blanco and Rio Obispo, the line might be run to a point opposite Cordoba, with very little bridging or culvert masonry. This route would involve a considerable deflection to the south, and, therefore, might increase the length of the line from Vera Cruz to Cordoba by at least 10 miles.

"It having been thus determined: First, that the Maltrata Pass, was the best one for the railroad for many miles south of the Peak of Orizaba; second, that the line must pass through the defile of Ingenio; and third, that a line passing through the gap where the River Atoyac breaks through the Chiquihuite Mountains would offer the shortest practicable line between Vera Cruz and Ingenio.

"The party received instructions early in April, 1858, to adopt that portion of the San Juan Railroad lying between Vera Cruz and Tejeria, and, from this initial point, run a line over the most favorable ground to the Atoyac Gap in the Chiquihuite Mountains, thence to Cordoba, and through the Ingenio Valley to the Maltrata Summit.

"Previous, however, to the location of this line, an experimental line had been run from Camaron to San Juan de la Punta, and also reconnaissance made of the country in a straight line between La Soledad and San Juan de la Punta, all of which went to prove that the country between the points referred to was very much broken and nearly impracticable for railway purposes.

"The line located in obedience to the above instructions diverges from the San Juan Railroad ½ mile west of the Tejeria station on that road, and runs thence in a southwesterly direction and in a very direct line, passing a little to the north of Rancho Nuevo, Mata Cordera, La Purga, Arroyo de Piedra, Mata Judio, to the considerable village of La Soledad, where it crosses the River Jamapa by a bridge, 102 ft. high and 150 ft. long. The first 6 miles of this line crosses low sand hills and valleys, which, although covered at that time with verdure, were once the shifting sands blown up from the

Gulf of Mexico. At the end of these 6 miles, the line enters on ground of a new and peculiar formation, called, in Mexico, 'tepetate', a name applied to the drift or deposit of detritus, the débris of the rock which forms the materials of the Cordilleras. This detritus extends to an immense depth. In the first 10 or 12 ft. from the surface there are water-worn stones and sand cemented together; below this, the material is composed of gravel, then sand, and, lower still, sand of a very fine quality. This tepetate rock is soft when first removed from its bed, but when exposed to the sun for a few days becomes harder, and might be used as a building stone in places where it would not be subjected to great pressure. When first quarried it is easily pulverized, and makes an excellent constituent for mortar or cement.

"The line from Tejeria to La Soledad, as before described, is undulating, the heaviest grade at any place being 60 ft. per mile. The line after crossing the Jamapa River at La Soledad occupies the south bank of that stream. Thus availing ourselves of a most remarkable natural feature in this section of country, which is that the southern bank of the Jamapa River is the ridge of the water-shed from which the streams fall into the Rio Atoyac, so that by running the line as near the bank of the river as practicable, and occupying the plain at the point where it began to resolve itself into ridge and valley, a way was found for the roadbed already prepared by natural means. The line, therefore, runs from the crossing of the Jamapa for a distance of 17 miles near the bank of that river, the work of graduation and masonry on that portion of the line being exceedingly light; the grades, however, are heavy, rising at first 70 ft. per mile, then 80, 85, 90, and finally running up to 110 ft. per mile. The line has here reached a point 36 miles from Tejeria, where the Jamapa runs no longer in our true direction, but takes a northwest course to Pueblo Viejo. The line at this point leaves the bank of the river, and runs southwest, crossing a stream at Paso Tablo and encountering near the Rancho Santa Teresa, the Rio de San Alejo, an inconsiderable stream at this point, running in a *barranca* of no great depth. From the crossing of the San Alejo, the line descends to the Canalita, a small stream which takes its rise near the Hacienda Defensa and runs along the eastern base of the Chiquihuite range. The line crosses the Canalita, at an elevation of 90 ft. and thence it occupies the slope of the Chiquihuite Mountains to the Gap of the Atoyac. From the crossing of the Jamapa River at La Soledad to the crossing of the San Alejo at Santa Teresa the line is comparatively open and free from woodland, if we except a few groves of trees which here and there dot the plain; not so, however, the ground between the Rio San Alejo and the Chiquihuite Mountains; this country is thickly wooded, with underbrush below, and parasitical plants and creeping vines hanging to the branches of the trees above, in places almost excluding the light and rendering it a matter of extreme difficulty to discover the best route, and to cut a way through it. The Chiquihuite Mountains are densely wooded. Many of these woods will be available for the railroad. The live oak which here abounds will form a lasting sill, and fuel for a century to come can be obtained on these mountains.

"The line, as above stated, occupies the eastern slope of the Chiquihuites, from the crossing of the Canalita to the gap made by

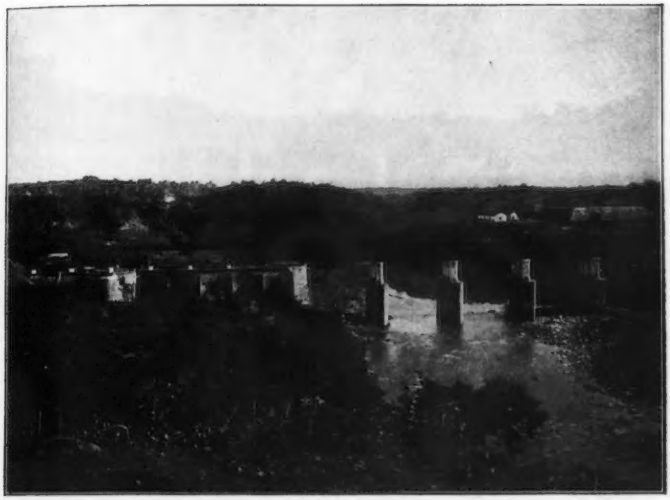


FIG. 3.—SOLEDAD BRIDGE, OVER JAMAPA RIVER, 42 KM. (26 MILES) WEST OF VERA CRUZ. ORIGINALLY A HIGHWAY BRIDGE COSTING $328 109. OVER THIS BRIDGE MR. M. E. LYONS BUILT A WOODEN STRUCTURE COSTING $200 000, WHICH LASTED UNTIL 1868, WHEN IT WAS REPLACED BY AN IRON BRIDGE.

FIG. 4.—SAN ALEJO BRIDGE, OVER THE RIO DE SAN ALEJO, 82 KM. (51 MILES) WEST OF VERA CRUZ. 100 METERS (328 FT.) LONG.

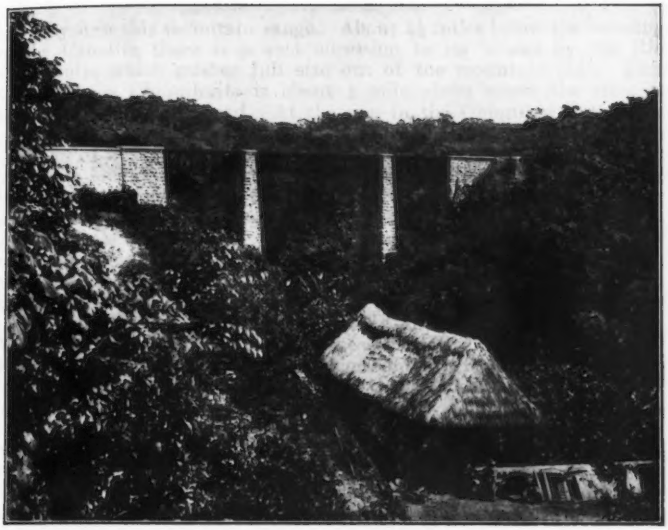


FIG. 5.—CHIQUIHUITE BRIDGE, OVER RIO CHIQUIHUITE, 83 KM. (51½ MILES) WEST OF VERA CRUZ. 67 METERS (220 FT.) LONG.

FIG. 6.—ATOYAC BRIDGE, OVER RIO ATOYAC, 86 KM. (53½ MILES) WEST OF VERA CRUZ. 100 METERS (328 FT.) LONG.

the Atoyac in this mountain range. About 1½ miles below the crossing of the Canalita there is a vast accession to its waters by the Rio Chiquihuite which gushes full size out of the mountain side. This source of the Chiquihuite is about ¼ mile above where the river is bridged by the public road. At the gap in the Chiquihuite, the line crosses from the north to the south bank of the Atoyac River, at an elevation of 304 ft. above the water. It will be possible hereafter to lessen the height of this bridge by increasing the grade—now nearly level—on both sides of the Atoyac River, and to introduce such other modifications of the line as future explorations may prove advisable. About a mile west of the crossing of the Atoyac the line leaves the mountain side and enters the Valley of El Potrero; then, occupying the divide between the waters of the Atoyac and the Seco, it passes through a densely wooded country with light earthwork, and but little masonry, till it reaches the Rio Seco, a short distance west of the Village of Peñuela. The ground rose so rapidly in the Valley of El Potrero, although but little indication of this fact would be given to the traveler with his unassisted eye, that grades of 132 ft. per mile became necessary. The line crosses the Rio Seco 60 ft. above the water, and then, curving to the south, intersects the public road a short distance west of Peñuela, and passing over some broken ground to San Miguelito, from which it runs to a point opposite the City of Cordoba, occupying a narrow ridge which offers both favorable ground and alignment. The grade from San Miguelito to Cordoba is 132 ft. per mile.

"An attempt was made to carry the road in a direct course from the point opposite Cordoba to Las Animas, but it was found that a grade steeper than 132 ft. to the mile would be required, and therefore it was deemed advisable, rather than increase this grade, to make a slight detour for the purpose of lengthening the line by way of Dos Caminos to Animas, and thence to the crossing of the Barranca de Metlac. It is probable that on final location this line would not cross the stream near San Miguelito, but would occupy the ridge nearer to the public road, and run closer to the City of Cordoba than the present line, which runs nearly a mile south of that city.

"The country is tolerably open from the crossing of the Rio Seco to the City of Cordoba.

"This city has a population of about 5 000 to 6 000. From its general appearance it seems to have had more prosperous days. It contains a very fine parochial church, several religious institutions, a barracks, some very good stores, and hotels. The soil in the vicinity seems to be very rich, but it is not much cultivated except in the immediate vicinity of the city, where vegetables, bananas, oranges, and coffee are raised. The plains which surround the city for many a square league are used for pasturage.

"The line has been described to the point where it crosses the Barranca de Metlac. This *barranca* is the most formidable obstacle to be encountered at any point between Vera Cruz and Orizaba. It is a deep gorge cut beneath the level of the plain. From the grade points on each bank, the distance is 1 085 ft. across, and the depth, from the grade to the level of the water in the bottom, is 346 ft.

"The subject of bridging this *barranca* requires considerable study,

and the preparation of plans and details in order that the subject may be viewed in all its bearings, and the question of economy and stability may be fully discussed.

"The erection of such a bridge will necessarily occupy much time, and therefore examinations were made, to ascertain if it were practicable to obviate the building of this high bridge, or to ascertain if some temporary expedient could not be resorted to, by which the road could be worked, before the high bridge was completed. A line with a descending grade, was run down the eastern slope of the *barranca*, and advantage taken of a point where a bend occurs in the stream, and where the valley widens, to cross the river at that point, and curving to the south on a radius of 400 ft., to the opposite bank of the *barranca*, ascending thence to the plain. This line would cross the river with a bridge 500 ft. long, and 140 ft. high, but it would add 2 miles to the length of the road.

"On gaining the western slope of the *barranca*, the line occupies a favorable valley, and, with the exception of crossing three or four streams, meets with no important obstruction until it reaches Cocolapan, and the City of Orizaba. From the Barranca de Metlac to Cocolapan the grades do not exceed 106 ft. to the mile.

"From the City of Orizaba westward the line runs through valleys bounded by majestic mountains, the presence of which greatly disturbs preconceived ideas of distance, for wide valleys are dwarfed in their vicinity to the appearance of one-third their actual width.

"The line occupies nearly the center of the Ingenio Valley, and west of the Town of Ingenio it curves to the north, and runs through the Valley of Encinal, passing a little to the west of the *hacienda* of that name. This valley rises so abruptly that it becomes necessary to introduce grades of 200 ft. to the mile; near this the head of the line curves to the south and enters on the slope of the *barranca* which leads from the Valley of Maltrata into that of the Encinal, and after encountering some heavy work, in rounding one or two bluffs, it emerges from this narrow gorge into the Valley of Maltrata at the north end of which stands the Indian village of the same name, containing a population of several hundred.

"The line of survey touches the north end of the village, where a light grade is introduced to enable the company to make a stopping place, and also to give the engine an opportunity to accumulate steam to propel the train up the steep grades which intervene between Maltrata Village and Maltrata Pass. From Maltrata the line ascends, with a grade of 203 ft. to the mile, for 3 miles, where another piece of light grade is introduced for the purpose before mentioned. The line has now reached an altitude of 6 100 ft. above the level of the sea; the valleys reach no higher, and the remainder of the ascent must be made on the steep and rugged sides of the mountain. Fortunately, there is only 8 miles of mountain line, a result certainly not looked for when we first viewed these formidable barriers from Tejeria.

"The ascent from the head of the Maltrata Valley to the Maltrata Summit is made with grades averaging 200 ft. to the mile, increased on straight lines to 212 ft., and proportionately reduced on curves.

"From the Maltrata Pass to the City of Mexico the railway can be carried over a series of plains, some of them separated one from an-

FIG. 7.—METLAC BRIDGE, OVER BARRANCA DE METLAC, BETWEEN CORDOBA AND ORIZABA, NEAR FORTIN, 116 KM. (72 MILES) WEST OF VERA CRUZ. NINE SPANS OF 50 FT. EACH. 80 FT. HIGH.

FIG. 8.—WIMMER BRIDGE, ON THE MALTRATA "INCLINE". 170 KM. (105½ MILES) WEST OF VERA CRUZ. FIVE SPANS OF 17 M. (55¾ FT.) EACH. "HORSE SHOE BEND" IN RIGHT DISTANCE.

FIG. 9.—INFIERNILLO VIADUCT, EAST OF MALTRATA, 150 KM. (93 MILES) WEST OF VERA CRUZ.

FIG. 10.—PANORAMIC VIEW, HEIGHTS OF MALTRATA. VOLCANO OF ORIZABA IN THE DISTANCE.

other by low trap dikes, others, again, are regular steppes, one plain being at an elevation of from 200 to 300 ft. above the other, but the ascent or descent can always be made by running along the slopes of the mountains. The direct line between the Maltrata Pass and the City of Mexico would leave the important City of Puebla nearly 30 miles to the south, rendering necessary a branch line of that length to connect it with the main line.

"To obviate this, explorations were set on foot to ascertain the practicability of connecting the valleys of Puebla and Mexico, by a tolerably direct line. It was soon discovered that no practicable route for a railway lay in the vicinity of the public road near Rio Frio, which at its greatest elevation attains an altitude of 10 500 ft., or nearly 2 miles in vertical height above the level of the ocean. Explorations were also conducted in the vicinity of San Martin to ascertain the practicability of finding a route for a railroad between that point and the Hacienda San Nicholas. This line would involve the building of 14 miles of mountain line, with a uniform grade of 106 ft. per mile. It was found, moreover, that the saving in distance would not be very great, as considerable northing would have to be made, in order to turn the chain of high mountains near Apam. A line running along the base of the western slope of Mt. Malinche, and near the banks of the Rio Atoyac, presented the best line for connecting by railroad the City of Puebla with the plains of Apam. The surveyed line, therefore, was run with the view of making the City of Puebla a point on the main line of railroad. The line descended with the grade of the plain from the summit of Maltrata to Cuesta Blanca, a distance of 8 miles, over ground that required but little preparation for the roadbed. At Cuesta Blanca the line enters on the slopes of the mountains which separate the plains of Esperanza from the plains of San Agustin de Palmar, and descends with a grade of 86 ft. per mile to the latter plain, and, skirting for a distance of several miles the northern edge of the plain, it leaves the Town of San Agustin de Palmar 2 miles to the south, the Town of Quecholas about a mile to the north, and runs close to the Towns of Acacingo and Amozoc. The mountain portion of this line, some 5 miles of it, is really the only objectionable portion, for, after reaching the plains of Palmar and Acacingo, it runs with most excellent alignment and light grades to the City of Puebla. The whole of this valley is thickly populated and well cultivated, and is interspersed with towns and fine *haciendas*. The phenomenon of whirlwinds and the mirage were made familiar to us here. At several points in the valley the whirlwinds may be seen lifting the light grains of sand and forming columns 60 ft. in height which are carried in every direction by the currents of air. Some of the gentlemen were much deceived in the appearance of water. The light reflected at a favorable angle from the sand gave portions of the plain the appearance of a sheet of water, a cloud would at times intervene and the phantom lake would instantly disappear.

"The surveyed line did not enter the City of Puebla, but ran a little north of the Guadalupe hill, and a short line was afterwards surveyed to connect the main line of survey with the plaza in that city. The line of survey runs thence along the base of the mountain of Malinche (the base of which extends over a diameter of 20 miles)

and intersects a great number of *barrancas,* the floods from this elevated peak having cut deeply into the slopes of the mountain. These *barrancas,* in many cases, were as deep as they were wide, and in some places they appeared to be so narrow, that it seemed possible to jump from one side to the other, and yet they were very deep. The upper portions consist of a multitude of channels spreading out with fan-like appearance. These *barrancas* obtain their greatest depth in a few hundred yards. In the rainy season the waters run with great velocity until they are somewhat checked by the flattening of the mountain slopes; they then begin to deposit the material they held in suspension, and from that point to the banks of the Atoyac, at the base of the mountains, these *barrancas* become shallower.

"The strata of the lower portion of the mountain are hidden from view by a thick deposit of detritus, which forms the surface and constitutes the material of this and all the other mountain slopes in the neighborhood of the line of survey between Puebla and the City of Mexico. The line, after running for nearly 30 miles due north from Puebla, enters the plains of Apam, and, running a short distance south of the town and lake of that name, proceeds for many miles over nearly level plains, with light grades, and still lighter earthwork, until it reaches the point where it begins to ascend to the Otumba summit. The elevation of this summit is 8 272 ft. above the sea, and, therefore, is the highest ground over which the line passes between Vera Cruz and the Capital. The excavation through the Otumba summit will be through 'tepetate'. From the Otumba summit the line descends into the far-famed Valley of Mexico, passing near the Town of Teotihuacan, Tepexpam and a number of villages and *haciendas,* and, running for 15 miles along the shores of the salt-water lake, Texcoco, joins the Guadalupe Railroad about 3½ miles from the City of Mexico and ½ mile from the northern terminus of this little road at Guadalupe.

"The surveyed line from Puebla to Mexico describes nearly the periphery of a semicircle the center of which would be in the neighborhood of Popocatepetl. It is 112½ miles from Puebla to the junction with the Guadalupe road, and 116 miles between the two cities. The grade descending from the plains of Otumba to the Valley of Mexico is 90 ft. to the mile, but this can be reduced, and also the grades at other points, by lengthening the line where these heavy grades occur. The line passes near the Towns of San Pablo, and Santa Ana, and some 2 miles east of the ancient City of Tlaxcala. The land here produces all cereals and fruit of the temperate zone, and the plains of Apam produce the finest specimens of the aloe, or maguey plant which supplies the Cities of Puebla, Mexico, and Toluca with pulque, in which article there is an immense trade.

"The great advantage, says Baron von Humboldt, of the maguey growing on plains elevated from 7 000 to 10 000 ft. above the sea, is that it is not affected by change of climate, and that its firm vigorous leaves withstand hail, frost, and drought. A vigorous maguey plant will produce 1½ gal. of pulque per day for a period of 5 months. The Indian calculates the produce of the maguey at 150 bottles, and the value of a plant near its efflorescence at from $4 or $5 to $8. The maguey is not only the vine of the Aztec, but its fibrous leaves sup-

plied him with paper on which he painted his hieroglyphics, its prickles which terminate its leaves served for pins and nails, and were used by the heathen priests for piercing their arms in their acts of expiation. They also obtained thread from the maguey plant, and its pulpy substance, when roasted, formed the sustenance of a large portion of the population.

"The City of Mexico, containing a population estimated at from 170 000 to 200 000, stands near the center of the Valley of Mexico. The valley is of elliptical form, the greatest diameter being 48 miles, and its least diameter 32 miles. One-twentieth of its entire surface is covered with water. The whole valley, except to the north, where the railway enters it, is shut in by high porphyritic mountains.

"On mature deliberation, it was concluded to run another line from the Maltrata Pass, by way of Nopalucan and Huamantla, to join the line run by the way of Puebla on the Plains of Apam.

"This section of country was favorable for a railroad route, and if it should be finally adopted for the main trunk line, giving Puebla a branch line, it will save 19 miles in distance to travelers between Mexico and Vera Cruz over the line passing by Puebla. It will not add anything to the distance to be traveled between Puebla and Mexico, but will add 39 miles to the journey to be performed between Puebla and Vera Cruz.

"Detailed estimates of the cost of these lines, and tables of grades will be submitted in the latter portion of this report."

It will be noted that the first line of survey was through Puebla, placing that city on the main line, which route would be Vera Cruz, Boca del Monte, Maltrata Summit, Cuesta Blanca, Plains of Agustin de Palmar, Acacingo, Amozoc, Puebla, Mexico.

Later, a line was run by way of Nopalucan and Huamantla, a more northerly survey, joining the Puebla-Mexico line at what is now Apizaco.

Capt. Talcott, in his report, page 1556, apparently left the decision of the route to Señor Escandon, although he must have expressed his preference for the Puebla direct line, as shown by the following letter:

"MEXICO, April 18, 1859.

"Col. ANDREW TALCOTT, New York.

"MY DEAR COLONEL: Your letters of the 21st February and 7th March were received on the 9th of this month * * *, and also specifications of the two routes, such as you suppose the Decree of the 31st August requires of me. You are right in supposing there is no need of presenting the maps to the Government as long as the contract is not fulfilled on its part, but, in case of doing so, I see you are of the opinion that the Puebla line should be preferred to the northern line which renders a branch to Puebla indispensable * * *.

"I remain, my dear
"Col. Talcott,
"Yours very faithfully,
"ANTONIO ESCANDON."

APPENDIX B

TRANSLATION OF REPORT OF MR. PASCUAL ALMAZAN.*

"The geodetic positions of the points from which observations have been made, and of others which have been sighted upon from these, also the altitudes determined by barometric leveling, are already included in the map and profiles presented to Mr. Talcott, and for this reason I will not speak of them, except to remark briefly as to the position of Vera Cruz.

"This had been ascertained at first by its longitude and latitude as computed by Baron von Humboldt, but having computed the positions of La Soledad, Arroyo de Piedra, Santa Ana, Matacordero and Rancho Nuevo by the triangles formed with the Orizaba Peak and El Cofre de Perote, respectively, and having combined the positions so obtained with the data of partial distances and directions, the result was that, to make each of these agree with Vera Cruz, it was necessary to apply to this port a correction of nearly 1′ 4″ in arc with respect to its longitude calculated from Mexico City.

"Undoubtedly, the position of Vera Cruz is more correctly computed than that of the capital, as it is based on many data presented and discussed by Oltmans in the astronomical observations of Baron von Humboldt; therefore, the final correction with respect to the Paris or Greenwich meridian should be made on the longitude of Mexico, although in the maps in which the starting point is the meridian of this capital, as is the one made for the Escandon Railroad, the modification would be made on Vera Cruz.

"The electro-magnetic telegraph being now established between both cities, it is easy to determine by it, with all the accuracy of which the determination of local time is capable, the distance or difference in longitude between these two cities and in the same way that of several intermediate points, so as to have verification data.

"Passing now to the main object, I must say that several of the reconnaissances made have only shown a negative result, that is, it has been found to be impossible to carry the railroad through the reconnoitered places. The time spent in these reconnaissances, nevertheless, may be deemed as not wasted, because it saves the time that (had these not been made) would have been spent in taking the topography and levels of the lines, which may now be deemed as impracticable.

"In this category is that of Pueblo Viejo and Ixhuatlan south of the River Jamapa, because, the direct pass from Ixhuatlan to Orizaba not being possible, it would be necessary to go up to Coscomatepec and come down immediately to the last named city [meaning Orizaba, probably] some 400 meters. This supposes:

"1st. The necessity of passing through Orizaba (if this line is adopted), which is absurd, as the only available pass for a railroad is the Garganta del Ingenio; and

"2d. That it is possible to build a grade for the road in the stretch between Pueblo Viejo and Ocotitlan, which is really impracticable if we consider the narrowness of the pass, its sinuosities in both horizontal

* A Mexican engineer who made some reconnaissances of the Jalapa route previous to the arrival of Capt. Talcott in Mexico.

and vertical directions, and the impossibility of widening at a reasonable cost a road made in the most broken part of Sierra del Chiquihuite, an upheaved limestone formation with the probable opening toward the north, and which, therefore, presents at this place a fractured edge.

"If the route through Orizaba is selected two lines are presented for reaching this city: One which almost coincides with the present road, and another passing south of San Juan de la Punta, which may pass through Cordoba or Zapoapita. If the first is taken, because special study indicates the possibility of a passage through the hills between La Peñuela and Omealca, there will be the advantage of a cheap crossing of the Metlac River, but only at the expense of attaining the height represented by the Tuxpango slope. This will certainly involve heavy work, but it is probably best to cross at this point.

"It may be possible, if the Chicahuastla foundation is not too brittle to overcome this height as indicated in the attached figure, by giving to the route the maximum slope of the Semmering.

"To cross the Metlac River farther up than El Fortin, it is necessary to choose the route of La Peñuela and Cordoba, and the selection would probably depend on the comparison of costs respectively computed for the two routes, but the fact that passing through Zapoapita will save more than a league (4 240 meters) in the distance from Peñuela to Orizaba, should be taken into consideration.

"The selection of another line that would coincide frequently with the National road and would go through La Peñuela, or perhaps through the south of El Potrero and Amatlan, might be directed to Zapoapita or Zacatepec in the above named manner, depends on the detailed inspection of the gorge of Atoyac between El Chiquihuite and El Potrero.

"It may be possible, by turning to the north of Tres Encinos and Paso del Macho, to make with relative ease the crossings of the small rivers of San Alejo and Chiquihuite, and passing along the west bank of this, follow parallel to its course in a south-southeast direction until reaching the gorge, and there pass to the eastern bank of the Atoyac, crossing it at the easiest place to pass to the west bank and enter the lands of Potrero.

"It is truly difficult to execute these plans, and I only indicate them with the greatest reserve, because I wish to show that the railway would be shortened by about 4 leagues, which would be needed if it was attempted to go around by way of Cotastla.

"In this way the Jamapa would be crossed a short distance to the west of Medellin and taking the south bank of the river above mentioned, the point above mentioned to the north of Tres Encinos would be sought.

"Having passed the gorge of the Ingenio (for which no other pass can be substituted), the heights of Maltrata and of Aculcingo are seen to be the best for making the ascent; the first presents the advantage of the least distance to Morelos and the difficulty of passing from El Encinal to Maltrata through a deep and long gorge; the second invites because of its easy access to Aculcingo, at the foot of the mountain chain, and because in its vicinity (as may be seen on the map) is the Reyes ravine, the lowest pass for crossing the mountains, especially if a small tunnel is opened, thus further diminishing the altitude.

"If this route is selected, it would probably continue through Chapulco, to Carmen or to El Camero and other points that will be shown on the small-scale map that is being prepared.

"The line from San Antonio de Arriba where the barometer indicates 511 mm. is, of course, impracticable.*

"The river of Santa Catalina which belongs to the western slope of the mountain ranges and begins about 3 leagues south of the *hacienda* of San Diego would make the route of considerable length, and has at its highest point about the same altitude as that of Aculcingo, and, after ascending it, it would be necessary to descend about 500 meters to the south, only to go up the same distance by the Tehuacan route.

"It seems, therefore, that this line should not be examined.

"Between the bed of the Jamapa River and the National Bridge there are no favorable routes by which to ascend to the central plateau, and, for this reason, the examination of this region could be useful only if an easy access was found to reach a considerable height, and if, furthermore, it were easy to pass from this point to the route to be selected in the direction of Orizaba or Jalapa; but this second condition cannot be considered, because, whether running north or south, it is necessary to cross the many rivers and ravines of great depth that descend from the Cofre and Orizaba peaks, respectively.

"There remains, therefore, only the line through Jalapa, which on the National Road presents the considerable ravines of Paso de Ovejas, Puente Nacional, Plan del Rio and Dos Rios, the difficult roundabout route of Cerro Gordo, and an extremely broken section from San Juan and El Zopilote to Tolome. I believe this line also should not occupy the attention of the engineers.

"This does not apply to the Actopan River which can be reached by describing a quadrant from the west of Antigua up to the Actopan Bridge, crossing there the river of this name to take advantage of the plain of this same town, recrossing the river in front of Saetal and continuing to the west on the course of the Abas or on the adjoining argillaceous formations until reaching the height of Las Vigas.

"This line presents the advantages of a continuous ascent without the necessity, in any case, of losing any of the altitude gained; the advantage of being on the route that most directly, and therefore, with the least length of line, leads toward Mexico; and last, the advantage of crossing the State of Vera Cruz on lands of small value.

"The disadvantages of this route are that it deprives the railroad of the commercial branch to Oaxaca and of frequently having to form the roadbed on lava formation where cuts are difficult and would constantly require filling, thus losing the advantages to be had in railroad construction when a line can be run where the cuts are equal to the fills.

"There is also another feature that might endanger the stability of the road. It seems that the lava formation is submarine, that is, the outside was suddenly chilled so that the crust was unable to contract, and when the interior of the current diminished in volume, many cavities were formed, frequently very near the surface, which in places have been uncovered by the mechanical breaking of their light shells

* The altitude was not computed because, the barometer being out of order, its indications were not reliable.

and in places form cavities of such volume that they absorb all the waters that flow from the slopes of this long valley. It is true that these disadvantages may be avoided by running over the argillaceous deposits at the sides, but even thus it will be necessary to cross the unstable strata for lengths totaling perhaps 6 000 meters.

"I believe the preceding indications are sufficient to enable the chief engineer to know what reconnaissances should be made in order to obtain the necessary data for deciding on and laying out the route.

"I will only add that, having had the opportunity to compare frequently the oscillations of the barometer in Vera Cruz with the simultaneous ones of another instrument in a different place, I have observed that the meteorological disturbances were not equally recorded by both instruments.

"The sinuosities and configuration of the country, the relation of different places in the valleys of the streams, make unnoticeable the small variations that may be seen in their full intensity at a place as open as Vera Cruz. This circumstance has given me, for the same place, altitudes that vary by more than 30 meters, and for this reason I believe it is necessary, hereafter, to try to diminish this cause of error.

"It may be advisable to place barometers for comparison in three different places, so as to compare the observations made in three different zones. The one at Vera Cruz would be used as a reference for the observations from the coast to the line from Paso del Toro in the valley of Actopan to San Juan de la Punta and Trapiche de Mesa. That at Orizaba would be used from this line to the crest of the range, and that of Mexico (or perhaps that of Puebla would be best, as it is a central point of the region) would provide the data for the computations of the central plateau.

"In this way the barometer, which is a meteorological rather than a mathematical instrument, would probably approach the former more nearly, because the non-computable variations which are due to atmospheric phenomena would be in part eliminated.

"I close this brief description of the local data because the scientific facts are all available to the Chief Engineer.

"ORIZABA, JANUARY 9, 1858." "P. ALMAZAN."

In 1864, Sr. Almazan was employed on construction work under Capt. Talcott.

In a letter written by Sr. Escandon to Capt. Talcott, dated Paris, 1st September, 1864, occurs the following passage:

"What you say concerning the right of way agrees with what I wrote you myself on this subject in my last letter. I think it would be well for you to withdraw altogether Mr. Almazan from the works now entrusted to him, and to set him with Mr. Merino to do the needful for securing the lands required by the line, which may be done now, if properly managed, with very little and in most cases with no expense at all. Any purchase of lands or agreements will be signed in my name by Mr. Elgnore, my attorney.

"ANTONIO ESCANDON."

By referring to Table 1, it will be noted that the cost of general management, real estate, land damages, etc., is not given, as there were no data for estimating these items.

APPENDIX C

Letters from Mr. William R. Eastman,* Transitman on the Original Surveyed Line for the Maltrata Incline.

"I have read with great interest the report of Capt. Andrew Talcott regarding the location of the Mexico and Pacific Railroad in 1858. I was one of the engineering party brought by Capt. Talcott from the United States, and landed with them at Vera Cruz on January 4th of that year. I was in charge of transit work for one of the field parties, and, in that capacity, ran the experimental line around the Maltrata Valley in July following. In searching for a way from the City of Orizaba to the high table-land, involving a rise of 4 000 ft., within a distance of about 16 miles in a direct line, the ascent at Maltrata was not the first to be tried. At that point there was no wagon road, but, instead, there was an excellent path for horses and pack trains reaching the summit by numerous zigzags. All wheeled vehicles, however, including the daily stages to and from the Capital, were obliged to climb the mountain by the *"camino real"*, the broad national highway which led up through Aculcingo, some 20 miles to the south. As this route had been chosen for the wagon road, the obvious presumption was in its favor, and the pass at Aculcingo was first examined.

"This was done in February with great thoroughness by two field parties setting out from a common point on a level shelf more than half way up the slope and working thence in opposite directions, one up and the other down. Neither party succeeded in making a satisfactory connection, either with the summit or with the valley below. The result was also disappointing with regard to distance and probable cost of construction, and the attempt to locate at this point was definitely abandoned.

"Before the mountain problem was again taken up, in the following July, there had been a diligent search for a better route, and it was found that the pass at Maltrata offered certain positive advantages.

"In going by Aculcingo the stage road made a considerable detour to the south. This elbow would be cut off and the distances to Mexico materially shortened by going up at Maltrata. Besides, there was, on the north side of the Maltrata Valley, a mountain wall in the form of a buttress or spur jutting out squarely to the east from the summit of the pass and, at a distance of 5 or 6 miles, turning north, bounding from that point a transverse valley parallel with the main crest of the table-land.

"The sides of this great mountain spur, lying directly along the general course which we desired to follow, afforded abundant space for prolonging or shortening a railroad line so as to fit the particular grade which might be taken as a maximum. Capt. Talcott, in his report, states very clearly his views respecting a maximum grade, and the topography of Maltrata suited his purpose.

"My party began at the summit at Boca del Monte to cut a line to the valley, which was in full view about 2 000 ft. below. We could make no use whatever of the steep zigzag trail. My instructions were

* Now living in Albany, N. Y.

to set the transit at a depression of about 4 ft. in the hundred and to follow the mountain side wherever the line might carry me. Progress was slow. Three or four men were required to cut an opening through the dense growth of small trees, bushes, and vines, so as to permit a sight from one angle to the next and to allow measurements with the chain. A leveler accompanied the party to note and correct the grade.

"Twice it was necessary to triangulate across abrupt ravines. After some 3 weeks of difficult work we came out on the floor of the valley with field notes for about 11 miles of line, substantially the same, at least in general outline, as that on which the road was built, and which is still shown by the railroad in a printed diagram on the time-tables of the company.

"When the maximum grade for such an incline was determined there could scarcely have been a better location for the railroad ascent to the table-land.

"WILLIAM R. EASTMAN."

"ALBANY, N. Y., MARCH 6TH, 1915."

THE SURVEY OF A ROUTE FOR THE MEXICAN RAILWAY.
BY WILLIAM R. EASTMAN.

"The Mexican Railway, connecting the City of Mexico with Vera Cruz, was opened for public use on January 1st, 1873. The completion of this line between the capital and the nearest and most important seaport was recognized as an event of national importance, and the President of the Republic, Lerdo de Tejada, led the public rejoicings. The road was 264 miles long, had been built at great cost, and represented the planning, working, and vexatious delays of at least 20 years. M. Romero* places the extent of time required at 36 years. From him we learn that a concession for a railroad to Vera Cruz was granted in 1837 and another in 1842. Notwithstanding large appropriations made by the Government, only 11½ miles were built up to November, 1850. At the time of the United States' invasion, in 1847, there were no railroads.

"*The First Railroad.*—The first 15 miles were built from Vera Cruz through Tejeria to the small village of San Juan. In 1858, cars on this road were drawn by a locomotive made in Belgium. In 1854, a short experimental railway, a sort of miniature model of a railroad, was set up in the Valley of Mexico, running out about 3 miles to the suburb of Guadalupe, where was a famous shrine of the Virgin to which frequent pilgrimages were made by all classes of people. This road was wholly of American construction, the rails and other equipment having been hauled 260 miles from the coast. The locomotive was built in the shops at Paterson, N. J., and brought up in detached pieces which were put together by American mechanics. This little road was convenient and popular.

"In 1857, a new concession for a railroad to the coast was obtained from the Mexican Congress by Don Antonio Escandon, a wealthy owner of mines and factories, who paid $1 000 000 into the national treasury and at once entered upon the undertaking with an earnestness worthy of his purpose.

* See his article in the *International Review* of November, 1882, Vol. 13, p. 480.

"*A Mountain to be Climbed.*—The route was evidently difficult, because there was a mountain which must be climbed. The broad table-land of that part of Mexico is, in round numbers, 8 000 ft. higher than sea level. This great elevation is reached in the first 100 miles from the coast. If the slope of the land were uniform and unbroken, an average rise of 80 ft. for each mile, while troublesome, might not be thought a serious obstacle; but the ascent is neither even nor uninterrupted. Starting off with an easy rise, the way gradually grows steeper and steeper. It takes the surveyor by surprise, when, on ground that looks to be almost level, his instruments tell him that he is 40, and then 50, and then 60, ft. higher at the end of a mile than he was at the beginning. At 95 miles, he has risen to 5 500 ft. Then, suddenly, in the distance of 5 or 6 miles, as the wagon road runs, the mountain has lifted itself 2 500 ft. higher still, up to the *cumbres* or sharp edge of the table-land which stretches away in a nearly level plain for 400 miles. This abrupt mountain wall was the railroad problem.

"*Ravines to be Crossed.*—There were also difficulties from deep and steep ravines or *barrancas* which cut across the path. One in particular, the Barranca de Metlac, hid a rushing stream 300 ft. below the surface of the surrounding fields. There was also a line of little hills, the Chiquihuites, standing as a barrier directly across the line of approach. Such were some of the natural obstacles, and it was evident that the best professional advice would be essential to success.

"*An American Engineer and his Party.*—This help was secured by the engagement of an eminent and experienced American engineer, Capt. Andrew Talcott. He was a graduate of West Point, and had rendered conspicuous service in the United States army for 20 years. During that time he had been engaged on such works as the fortifying of Hampton Roads, the building of Fort Hamilton, in New York Harbor, the work at Fort Adams, in Rhode Island, and the improvements of the Upper Hudson River. Resigning his army commission, he had been in demand for boundary surveys in Maine, Michigan, and Iowa, and had been chief engineer of two important railroad systems. He was well fitted by training and experience for this new task.

"A surveying party of some thirty men was at once recruited in the neighborhood of New York, several of them being taken from the staff of the Croton Water-Works and others from the Pennsylvania and Reading Railroads. The chief assistants were M. E. Lyons, of Reading, Pa., and R. B. Gorsuch, of New York, the latter having already had experience in Mexico in connection with the building of the experimental railroad to Guadalupe. Camp equipment, instruments, etc., were shipped from New York.

"The main party sailed by steamer from New Orleans, on January 1st, 1858, and landed at Vera Cruz 4 days later. A camp was immediately established at Tejeria, 9 miles out from the city, and the working parties were organized. These were three in number, a headquarters party, to explore the country on horseback, and two field parties, each with transit and level, prepared to do exact work and keep careful records. In view of the uncertain character of strangers who might be abroad, the camps were always guarded by a sentry at night, and by day each American wore a revolver in his belt.

"Roads and Routes of Travel.—At that time, in Mexico, as in our own colonial days, land travel, if not done on foot, was chiefly on horseback. Merchandise was packed on mules. Wheeled vehicles, except in the cities, were few. Although there were trails and bridle paths in every direction, there were no roads possible for carriages except the one thoroughfare, the *camino real,* from one large city to the next. From Vera Cruz inland there were two such highways. One was by way of Jalapa, mounting the table-land on the north side of Mount Perote, beyond the great snow peak of Orizaba. This route was chosen by General Scott with the American army in 1847. The other road, somewhat more direct, passed through the City of Orizaba, climbed the mountain pass at Aculcingo to the south of the snow peak, and joined the Jalapa road at San Marcos, many miles to the west, leading on thence to Puebla and Mexico. This was the route taken by Cortez and his Spaniards. It was the regular stage route in 1858, and was adopted as the base of the survey. In many respects this was a remarkable road, built probably by the Spaniards 200 or, perhaps, 300 years ago. The location of this broad highway through a mountain country showed great skill in the builders. The massive, arched stone bridges were still intact, and as strong as when first built. There were in many places the remains of a stone pavement; but years or centuries of neglect and spoliation had left only scattered blocks of granite which were more of a hindrance than a help to passage. Along this national road, mule trains with packs of merchandise were constantly passing, up and down, but almost the only wheeled vehicle that used the road was the *diligencia* or stage coach which passed daily in both directions.

"The journey from the coast to the capital required 3 days. At Vera Cruz the coach—one of the familiar type which we know as a Concord coach—was placed on a platform car and went by train 9 miles to Tejeria. At that point horses were attached. There were never less than six horses and, on the steeper grades, eight and ten were often used. Relays were taken every 10 miles and, where the going was good, the team was driven at a gallop. The first day's trip was 82 miles to Orizaba, arriving in time for dinner, a quiet evening, and a part of a night's rest. Starting again before daylight, a distance of more than 90 miles to Puebla was made on the second day. The third day's ride of about 80 miles, passing over an elevation of 10 500 ft. above the sea and thence running rapidly down, brought the tired passengers into Mexico in the afternoon.

"Robbers on the Road.—The journey was tedious enough, but the magnificence of the scenery in the mountain passes was to many travelers no slight compensation. The trip was also subject to some thrilling adventures. The road, especially that part of it between Orizaba and Puebla, was infested with bands of robbers. It was the common daily experience of stage passengers to be stopped once or twice in the early morning and invited, with profuse apologies but at the point of a pistol, kindly to alight from the coach, while three or four men coolly proceeded to possess themselves of money or any valuables that might be found on the persons of the travelers or concealed under the cushions. Articles of clothing, such as hats, coats, and shoes, were often taken from the wearers, and silk dresses

were sometimes stripped from women. If so fortunate as to find a priest among the passengers, the bandits would persuade him also to contribute by holding a pistol at his head until he pronounced a full absolution for the crime.

"The leaders of these robber parties were often sons of wealthy landowners who found life dull on the *hacienda* and led these forays for the mere excitement or the fun of doing it. Passengers, on the other hand, went prepared to pay a little toll, considering it a custom of the country, and carried only a few silver dollars and an old silver watch, and wore their old clothes when compelled to travel by diligence. The American surveying party, whether in its own camps or at its work, was not in any way molested. In September, 1858, however, when the preliminary survey was completed, a party of nine Americans found themselves paid off at the City of Mexico and were unable to obtain New York funds, except at a discount of 15 per cent. They therefore decided to engage the coach for themselves and, being well armed, to carry some $3 000 in coin to the coast. On the second day out, they were twice challenged in the usual way by robbers, and a lively fusillade followed in each instance. No one of the American party was touched, but the woodwork of the coach showed more than one bullet hole. One of the assailants was left dead by the roadside and two others were afterward known to have died from wounds received.*

"A surprising sequence of the above encounter was the sad fate of a Mexican who was traveling in the opposite direction on that unlucky day, and was alone in the coach when, along in the afternoon, it reached the spot where the American engineers had been in the morning. This solitary traveler, who had never heard of the engineers, was taken out by a band of ten men armed to the teeth, was stripped to his underclothes, tied to a tree and lashed without mercy. This was Mexican retribution.

"*Disturbed Conditions.*—The work of the two field parties, begun early in the year, had not continued 2 months before it became evident that the disturbed political and military conditions must seriously embarrass their work. Comonfort was President, and Zuloaga was contesting his claim. Detachments of troops from one side or the other were marching back and forth over the road, and in a small way were fortifying ground here and there along the line. Now and then the sound of cannon and volleys of musketry was heard. There was no very serious clash of arms, but there was just enough of the form of fighting to be annoying to peaceable surveyors. Consequently, the force was reduced, one field party was kept, and the others returned home. The remaining party then began in earnest to examine the most critical point on the line, the abrupt ascent of the *cumbres*.

"*A Choice of Mountain Passes.*—The stage route climbed the mountain by the pass at Aculcingo by a series of ten or twelve extended zigzags. A railroad could hardly do that. After one height was reached, there was another beyond it, and the attempted line of survey became hopelessly involved in a very tangle of hills. A month was

* A notice of this affair may be found by the curious in *Harper's Weekly* for October 23d, 1858, at page 679, under the heading of "Mexico; A Band of Robbers Punished".

spent in trying the possibilities of the ground, and the result was failure. Then the scouting party reported that a few miles to the north was another pass up from Maltrata, and, on the further side of it, a very long mountain spur thrust out for miles into the lower country, a kind of buttress to the table-land. Thither the party turned. They found no carriage road in the Maltrata Pass and the zigzag turns for horses and mule trains were even more numerous than at Aculcingo, but hope was fixed on that prolonged spur of the mountain the sides of which towered half a mile above the village and were as yet unexplored and covered with low trees and tangled brush. After climbing up the path to Boca del Monte, the 'mouth of the mountain', which stands at an elevation of 7 922 ft., the transit was set at a downward gradient of 200 ft. to the mile, and, with only this as a guide, the mountain side was explored. No account was taken of curves, bridges, or tunnels. The chopping out of the path was slow. Straight lines and their angles were carefully measured and noted. The leveling instrument was close behind. Another full month was spent creeping farther and farther along.

"At one point, now called the 'Devil's Balcony', the cliff is so precipitous that it seemed to hang above the streets of Maltrata Village, 2 000 ft. below. Winding on and on, the point of the spur was turned, and a transverse valley to the left was entered. At the end of that came another turn on a hillside to the right and, at some distance farther, another and another turn, until the line drawn on paper resembled the outline of a boot with the leg resting along the valley. At last, after 11 miles of sharp descent, the low level of the village was successfully reached; but there was more of it to come. The water drainage of the valley is through a narrow gorge, so wild that the local name for it is the *Barranca de Infiernillos*, or, in English, 'The Ravine of Little Hell'. Down this rocky gorge the line goes plunging on, turning and twisting again, until at last, in the Valley of Encinal, it comes out into the midst of smooth fields and 10 miles of a straight course brings the line to Orizaba. In 18½ miles, the descent is 3 000 ft. The average of that is 165 ft. to the mile, but many miles are on a grade of 200 ft. or even more. This route has now been in use for 40 years. The locomotives are of the twin, Mogul type, built by Fairlie, each with two boilers, which alternate in supplying steam for the desperate pull.

"*Across the Deepest Gorge.*—The second problem was the crossing of the Barranca de Metlac. As a straight line across was only 1 400 ft., it was a challenge to the engineer to throw a suspension bridge over it, or at least a tubular or cantilever structure with high piers and an enormous span; but the first cost of such a work would also be enormous, and the cost of maintenance very great, and neither of those things was done. Perhaps, if the steel construction of high piers had been as well understood 50 years ago as it is to-day, the straight line might have been secured; but, instead of that, the line, crossing the edge of the ravine, runs down along the inside bank on a narrow shelf for a mile or more up stream until it reaches a broader part of the ravine and then turns across on an almost semicircular bridge, about 100 ft. above the water, and goes back by a similar shelf on the farther side until it reaches the top again and continues on its way.

"The third point of difficulty at the Chiquihuite is met by a considerable detour away from the line of the highway, the railroad striking the side of the hill and rising until it joins the carriage road again, crosses it and passes at once by a high bridge to the farther side of the Atoyac River.

"Thus, step by step, each problem was successfully solved, and in September, after a little more than 8 months in the country, the party returned to New York, the line of survey was plotted, and the report made.

"*Further Delay and Final Success.*—For a long time, nothing more was done. A new grant from the Government was made in 1861. The French Army entered Mexico in 1862. To facilitate their movements they contracted with Escandon to build portions of the road. It was extended over the lower, open country to Paso del Macho, 48 miles from Vera Cruz. It was also carried from Mexico east to Puebla. During this foreign régime, 133 miles were constructed. A still more liberal contract was made with the Republican Government in 1867, and another in 1868. The cost of building was found to be unusual and extreme. Incredible as it may seem, the Government insisted that the road should be built from both terminals simultaneously, making it necessary to haul heavy material, machinery, and equipment hundreds of miles over rough mountain roads in order to build back from the capital. The work of construction of the mountain division, still under the supervision of Capt. Talcott, was taken up in earnest in 1864; but Escandon was compelled to seek outside pecuniary help, which he found in England, and it was an English company which finished the road in 1872—the first railroad in Mexico.

"In spite of its great cost, the need of the railroad has been so great that on the $46 000 000 invested good profits have been paid out of the heavy charges, at least up to within the past year. The road is under English management, and the impression is common that the English built it. The fact is that the survey, the location, and the plan of construction were American throughout.

"From the character of the road it will be seen how easily it can be obstructed by acts of war, by the destruction of bridges and blocking of tunnels; but let us hope that this will never be done. The road is itself a triumph over natural obstacles; it ought not to be ruined by human passions.

"ALBANY, N. Y., JUNE 19TH, 1914."

Mr. Eastman also states that in addition to the members of the corps of engineers given on page 1550, there were several members who came directly to Vera Cruz from New York on another vessel. Three members of the original survey party were killed in the Civil War: Maj. Sidney Coolidge, 16th Regiment, U. S. A., at Chickamauga; Maj. Phillip J. Kearney, 11th New Jersey, at Gettysburg; and Lt. Col. Howard Carroll, 101st New York, at Antietam.

APPENDIX D

Construction Operations, 1861–64, French Occupation.

The unstable conditions existing in Mexico at the time of the presentation of Capt. Talcott's report, which prevented for the time being the construction of the proposed railroad, have already been noted. Under date of Tacubaya, May 19th, 1859, Don Manuel Escandon wrote Capt. Talcott:

"I have set my shoulder to the wheel, and have no doubt of procuring the formation of a Mexican and Puebla Company, to commence at once the works between these two cities, crossing by the Llanos de Apam, which line later on would be continued from Puebla to Orizaba and thence to Vera Cruz."

The question of a joint intervention in Mexican affairs by the European powers of Great Britain, France, Spain, and Prussia was mooted in 1860, and on December 14th, 1861, a Spanish fleet appeared before Vera Cruz, which was at once occupied by the Spanish troops under General Prim. The French arrived soon after, and in September, 1862, more French troops arrived. In view of the unhealthiness of Vera Cruz, the convention of Soledad was concluded with the Mexican Government, permitting the foreign troops to advance to Orizaba.

No construction work was done by Señor Antonio Escandon under the grant of May 31st, 1857, and on April 5th, 1861, he obtained a new grant from the Mexican Government which materially modified the provisions of the former one.

Capt. Talcott received a call to return to Mexico in May, 1861, with which he was unable to comply, and suggested the employment meantime of Mr. M. E. Lyons, his former Principal Assistant, which was accordingly done, and Capt. Talcott did not return to Mexico until March 1st, 1862, having left Charleston, S. C., on January 9th, 1862, on the blockade runner *Carolina* for Nassau, and thence *via* Havana to Vera Cruz, where on arrival he resumed his duties in connection with the construction of the Mexico and Pacific Railroad.

The French, on their arrival in Mexico, found no railway, except the few miles of track operated by mules from Vera Cruz, which was used to transport passengers and their baggage about 9 miles to Tejeria, from which point transportation to the interior was over old Spanish highways that at many points were almost impassable during the rainy season; and their efforts were immediately directed toward the construction of a railway across the *Tierra Caliente,* in which they seem to have had the co-operation of the "Escandons", for entries found in Capt. Talcott's diary for February, 1863, indicate that, as

the representative of Antonio Escandon, he was supervising work in progress in Soledad.

It appears, therefore, that the demand for better facilities for transportation by the French Army was responsible for the construction of what is now known as the "Mexican Railway" from Vera Cruz to Soledad, where it crosses the Jamapa River, and practically determined the Orizaba Route for the Imperial Mexican Railway Company in 1864.

It was not until after the arrival of Emperor Maximilian (Vera Cruz, May 28th, and City of Mexico, June 19th, 1864) that Antonio Escandon made rapid progress in his negotiations with English capitalists which finally resulted in a contract, dated August 19th, 1864, between "The Imperial Railway Company, Limited", and Smith, Knight and Company, Limited, by which the last named Company undertook to

"complete and construct a Railway from Vera Cruz to the City of Mexico and a Branch Railway to Puebla, according to the plans, sections, and specifications in that behalf made by Mr. Talcott, one of the Engineers of said Company."

In anticipation of the conclusion of this contract, Don Antonio Escandon had written to Capt. Talcott as follows:

"ANDREW TALCOTT, ESQ.,
Mexico. "PARIS, 31 July, 1864.

"DEAR SIR:—I thank you for the information you give me concerning your operations on the portion of the line near Mexico and am happy to see that Messrs. Almazan, Merino and your son Richard are employed on the same. You must bear in mind, as I told you before, that no rails are to be placed on that part of the line and that you have nothing else to do but to go on with the earthworks without exceeding your estimate of 1858, nor anyhow the amount of $15 000 per month. Any such excess would be a dead loss for me; on the contrary, I would profit by any difference below your said estimates, in conformity with which these works will be reimbursed by the contractors.

"I must renew my observations concerning the settlement of account with the French Government should the question be brought before you. This settlement must be done in strict accordance with the contract made with the French Engineers. That is to say, from the sums advanced must be deducted the amounts received by the French and the customs dues up to Dec. 31, 1864, and those due to the Railway for carriage of the French Army, its baggage, etc. If any difference results against me, I will pay it forthwith in money on 1st January, 1865, and can admit of no settlement made on other terms than these.

"I will besides recommend you most particularly to reserve all my rights against the French Government in consequence of the losses sustained by the Company by cases of *force majeure*, such for instance as that which occurred when W. Lyons was mortally wounded by the

guerillas, who carried off a certain sum of money belonging to the Company and destroyed part of the road. (The illness of W. Lyons, as you know, delayed for some time the progress of the works), besides several other cases which are not present now to my memory.

"The principal injury we sustained was when the guerillas carried off all the workmen on the line with the cash in hand for payment of the same. This is the point that must be principally insisted upon.

"Yours,

"ANTONIO ESCANDON."

After the execution of the contract with Smith, Knight and Company, Don Antonio Escandon wrote a letter to Capt. Talcott, as follows:

"PARIS, August 31, 1864.

"Messrs. Smith, Knight and Co., the constructors, have had the good fortune to secure the services of Mr. Lloyd, a friend of Messrs. Gibbs, who has constructed an important Railway in Peru, where he has resided 10 years. This gentleman, with Mr. Samuel, will leave for Mexico probably by the St. Nazaire Steamer of September. He will remain only a short time there to see what are the materials that he has to purchase in Europe for account of the constructors, after which he will return to England. You must therefore get everything ready for the delivery of the constructed part of the line, which will be made immediately after the arrival of these gentlemen.

"I have asked the French Minister of Public Works to grant me an audience in which I intend to request him to withdraw entirely the French Control in consequence of the formation of the new Company. So I think you will soon be easy on that score.

"THURSDAY, 1st September, 1864.

"After writing what precedes, and at the last hour I receive your favor of the 29th July.

"By what I have already told you, you will see that it is very urgent that you should immediately go over to Vera Cruz to get the Schedule made of all the Stock belonging to the Railway, without distribution even of the light rails, which, by my contract, will be returned to me by the Company.

"My engagement being to deliver the road up to the Paso del Macho I need not tell you that no expense for Surveys, etc., must be made beyond that point.

"ANTONIO ESCANDON."

It is very evident from the foregoing that what is now known as the "Ferrocarril Mexicano" from Vera Cruz *via* Soledad and Camaron to Paso del Macho was built by Don Antonio Escandon under the concession for the "Mexico and Pacific Railroad," for which surveys were made by Capt. Talcott in 1858.

It also shows that this part of the road was built under the control of the French Government, in accordance with an agreement made

with French engineers, and that for this 77 km. of railway the Imperial Mexican Railway Company paid Don Antonio Escandon an agreed price after it had been completed at his cost from Vera Cruz to Paso del Macho.

It is not unlikely that this section of railroad, at the time of formation of the Imperial Mexican Railway in 1864, was in a deteriorated condition and required extensive repairs, which is no doubt the meaning of the word "refections" used in the following letter:

"Mexico, October 17th, 1865.

"Private and Confidential.

"Col. Andrew Talcott,

"Dear Sir: As I understand that you are going over the line to Vera Cruz, I have to request you to give your attention to the refections which have been made by Mr. Pogenstecker under Mr. Lloyd's orders in pursuance of my contract with Mr. Lloyd.

"I have already paid $200 038.57 for these works, and I should be much obliged to you to report to me what has been done and how much more remains to be done.

"With respect to that part of the refections which has been completed I wish to know how much the same work would have cost under the conditions in which the works of construction are paid to Smith, Knight and Company.

"And as the work now executed must be submitted to your approval before being accepted by the Imperial Company, I should be much obliged to you to tell me how the refections have been carried out. Do you consider it necessary that the remainder should be completed on so expensive a scale; and how much do you think it should cost, according to Smith, Knight's contract with the Imperial Company?

"I am, dear Sir,

"Faithfully yours,

"Antonio Escandon."

It also appears that Don Antonio Escandon constructed, in part at least, the line from the City of Mexico to Guadalupe and east of Guadalupe to the Llanos de Apam, for which work he was paid by Smith, Knight and Company, on the basis of Capt. Talcott's original estimate of its cost, for the construction of this part of the line was included in the Company's contract to build the railroad from Paso del Macho to the City of Mexico.

APPENDIX E

CONSTRUCTION OPERATIONS, 1864–66, UNDER EMPEROR MAXIMILIAN.

Immediately after the organization of the Imperial Mexican Railway, August 19th, 1864, extensive measures for the speedy construction of the unfinished portion of the railroad were begun, although the transfer of the concession held by Don Antonio Escandon was not approved by the Emperor Maximilian until January 26th, 1865, which approval fixed the time of completion of the entire line by April 30th, 1869. The chief contractors, Messrs. Smith, Knight and Company, were the builders of the Chilian railroad from Valparaiso to Santiago, and came from that country to Mexico. They had their own staff of engineers, Mr. S. Samuel, Chief Engineer in London, England, and Mr. William Lloyd, Chief Engineer in Mexico, Messrs. William Cross Buchanan and Alister Fraser, Division Engineers, Mr. Richard Ingalls, Superintendent, and others, all Englishmen. On the staff of Capt. Andrew Talcott, were his son, Mr. Richard H. Talcott, now of Albany, N. Y., Mr. T. M. R. Talcott, now of Richmond, Va., Sebastian Wimmer, M. Am. Soc. C. E., now of Albany, Minn., the late Gen. W. H. Stevens, formerly on the staff of Gen. R. E. Lee, Col. H. T. Douglas, now of New York City, and other American engineers.

Many of the latter had served in the Confederate Army during the Civil War (United States), and had secured positions in Mexico at the close of that war, in April, 1865. The chief contractors, Smith, Knight and Company, were afterward succeeded by George B. Crawley and Company. Mr. Sebastian Wimmer, after his arrival in Mexico, was placed in charge of the construction of the seven upper sections of the Maltrata Incline (so-called) on April 26th, 1865, and some time after was placed in charge of additional sections, lower down, near the Town of Maltrata. While in charge of this work he made a number of modifications and changes in the alignment which greatly improved it.

The following excerpt from Mr. Wimmer's diary is of more than ordinary historical interest:

"*Saturday, April 29, 1865.*—Emperor Maximilian arrived at Maltrata at 12 o'clock noon. I was introduced to him, as well as the other engineers, and had quite a chat with him. He wanted to know how I, a German, from Munich, Bavaria, Germany, came to be associated with the English party of engineers engaged on the Imperial Mexican Railway. I explained that I was a citizen of the United States, a practicing civil engineer for the past fourteen years (1851-1865), having arrived in New York city, June 2d, 1851, and when civil engineers from the United States were required for service in Mexico, I applied for a position on this railroad and then received an appointment on this important work.

"He told me that he well remembered Abbot Boniface Wimmer, my uncle (with whom I arrived in New York as above), and favored his project to establish in America 'The Benedictine Order'.

"We all had a fine dinner in the open verandah of the chief contractor's house (Smith, Knight & Co.), a native Indian brass band played in the open patio. Leo Thun, Major General of the Austrian troops, quartered at Puebla, was one of his party, also his private secretary Elvin, Minister Ramista, Prince Wittkenstein, a Russian officer, and others of his escort. Others present were Wm. Lloyd, Chief Engineer of Smith, Knight & Co., Mr. Richard Ingalls, Superintendent, Engineers Richard H. Talcott, Fred. Simons, my friend Rudolph Wieser, Mr. Wm. H. Burr, Dr. Manifred, Wm. Cross Buchanan, John Quinn, and others.

"A messenger from Orizaba arrived with a telegram for the Emperor, announcing that President Lincoln of the United States was killed on April 14th, 1865, in Ford's Theater, Washington, and Secretary William H. Seward and Fred'k. Seward wounded, by John Wilkes Booth, the assassin. This news at once stopped the dinner, as the Emperor wanted to put himself in telegraphic communication, and Orizaba was the nearest point, some 15 miles away. We all escorted the Emperor to Orizaba that afternoon."

Also this excerpt:

"*Monday, December 25, 1865.*—At Orizaba. Christmas Day. Hot and dry. Attended midnight high mass with Davison. At 8 A. M. all we engineers rode about 3 miles down toward Cordoba, to escort Empress Carlotta to Orizaba, who was returning from Yucatan to the City of Mexico. She arrived about 11 o'clock, seated in an open carriage with a court lady, and accompanied by a large number of French Zouaves on horseback, several hundred, plowing through six inches of dusty road, causing such a dust that we could not see one another, and covering the poor Empress and her lady companion so that it was awful to behold. They escorted her to the Cathedral for a grand reception by the authorities and thence was to receive us, but, being exhausted, she went to Bringas residence to rest and to receive us the next day.

"*Tuesday, December 26, 1865.*—Hot weather. I called with Ed. Melgar on Col. Rodrigues to arrange about our introduction to the Empress. She received us at Bringas. General Uraga (one leg lost in war lately) acted as introducer. Richard Ingalls was our spokesman. Richard H. Talcott and myself, Mason Peek, Maynadier, W. S. Davison, Wm. B. Cooper, and Slate were introduced. She spoke Spanish fluently, but she soon discovered that most of us were rather embarrassed and preferred English, and when Mr. Ingalls was not aware that the Empress spoke English, she answered that it was the second language she had learned and loved to speak it. We thence got along very well, and when Mr. Ingalls invited her to come on her way up to the city of Mexico *via* the *Las Cumbres* work, she replied that she loved to accept the invitation, as the Emperor had already told her of his delight of having been there last spring, but she had already received a telegram from the Emperor, who was in Puebla, to

come on at once, without further delay, and as a dutiful wife, 'I must obey' and thus be unable at this time to see for herself how far the contractors of the railroad had proceeded toward a soon completion, for you have seen what a terrible thing it is to travel in this country, and so exhausting, and asked us to hurry the work to completion."

On July 8th, 1866, the Empress Carlotta left Mexico on a visit to Napoleon, Emperor of France, to arrange Mexican affairs, never to return. Her husband, the Emperor Maximilian, as is well known, was executed June 19th, 1867, at Queretaro.

Mr. Wimmer's diary contains a daily minute description of every happening during his connection with this work, 1865-1867, affording unlimited material for the historian, but lack of space prevents giving any more excerpts.

His name is indelibly associated with the Mexican Railway by having one of the bridges on the Maltrata Incline named after him, perhaps the only record where a civil engineer has thus been honored. This bridge is shown by Fig. 8.

Another official prominently connected with the Imperial Mexican Railway was Mr. Thomas Braniff, formerly of Staten Island, New York. He had been with Gen. Meigs in Peru, on the Lima and Oroyo Railroad over the Andes.

Later, Mr. Braniff became General Manager of the Mexican Railway, and also the owner of a large cotton factory at Ingenio, a short distance west of Orizaba, accumulating a large fortune and becoming an influential resident of Mexico.

In the spring of 1866, the Imperial Mexican Railway Company became financially embarrassed, and Capt. Talcott had to obtain funds from Barron, Forbes and Company, of the City of Mexico, to meet engineering expenses in carrying out instructions he had received to prepare and submit final estimates of the work done so far by Smith, Knight and Company and their successors George B. Crawley and Company.

At that time the railway was in operation between Vera Cruz and Paso del Macho, 48 miles, and was opened to Apizaco, 87 miles from the City of Mexico, for freight service, January 21st, 1867.

APPENDIX F

Construction Operations, 1867–73, Under Republic of Mexico.

Capt. Talcott severed his connection with the Imperial Mexican Railway on February 10th, 1867, he holding at the time the position of Joint Engineer in conjunction with Mr. James Samuel, of London, England. Leaving the City of Mexico, February 23d, 1867, he arrived in New York City on April 5th, 1867. During his connection with this railway he had constructed 135 miles of road.

After the downfall of Maximilian and the withdrawal of the French Army, Señor Escandon applied to the re-established Government of the Republic, to revalidate his concession, and, notwithstanding he had been a staunch supporter of the French and of the enemies of the Republic, the Mexican Government, on November 27th, 1867, made a new contract with him, under more liberal terms than his former grant. This contract was again modified on November 11th, 1868, and the railroad was finally finished on December 31st, 1872, under the Escandon grant, 16 years after his first contract was made, and 36 years from the date of the first concession. It was formally inaugurated on January 1st, 1873, by President Sebastian Lerdo de Tejada.

The Puebla Branch was finished on September 16th, 1869, the main line to Paso del Macho, in 1864, to Atoyac, January 9th, 1871, to Cordoba, August 22d, 1871, to Fortin, December 8th, 1871, and to Orizaba, September 5th, 1872.

A great deal of misinformation has been published about the Mexican Government requirement of hauling rails and equipment overland. This requirement was a practical necessity, and was dictated by common sense, as the construction work on the table-land was comparatively light and the grading between the City of Mexico, Apizaco, and Puebla was finished 5 or 6 years before that of the sections lying farther toward Vera Cruz, between Boca del Monte and Paso del Macho, where the heaviest work on the railway occurs.

This is particularly emphasized by the fact that it was proposed originally to form a company to build first a railroad between Puebla and the City of Mexico, by way of the Llanos de Apam, which line, it was believed, would become so lucrative that it would soon be continued to Orizaba and thence to Vera Cruz. Had not the French invasion occurred, this would have been the procedure of construction, although the original plan was partly carried out, with this modification.

DISCUSSION

Mr. Mathewson.

T. K. MATHEWSON,* M. AM. SOC. C. E. (by letter).—The wealth of information, both historical and technical, contained in Mr. Low's paper will be a revelation to most engineers, even to those who have often passed over the Mexican Railway. That it was located, and to a great extent built, by American engineers will be news to those who have always heard it spoken of in Mexico as "The Queen's Own". The road has always enjoyed an enviable reputation for safety, in spite of the sharp curves and steep grades.

The Maltrata Incline, with its 4% grades and great curvature, would seem to offer an attractive field for electrification. The nearness of water-power and the absence of snow and sleet make the conditions favorable. It is the writer's belief that such a change in motive power has been considered by the company. It is his understanding that a masonry flume was built in one of the streams, possibly the Atoyac, in which to make measurements of the flow, under the direction of the late George Foote, who was at one time Chief Engineer of the road. Possibly the articulated Fairlie engines, with their short wheel base, make electrification less attractive. This may well be the case, as fuel oil from the Tuxpam oil fields, near-by, is now used in place of coal briquettes from England. Details of these engines would be interesting, as they are one of the unusual features of the line.

The description and history of this interesting railway could hardly be more complete, in spite of the fact that the disturbed condition of Mexico has made communication with the country very difficult at this time. The paper will be especially interesting to those who know and love Mexico.

Mr. Douglas.

H. T. DOUGLAS,† ESQ.—Mr. Low is to be congratulated on the complete description of the Mexico and Pacific Railroad, the location and construction of which furnish one of the best examples of bold engineering—especially from Tejeria to Boca del Monte—to be found on this continent.

The writer had the honor to serve under Col. Talcott in locating the line from Paso del Macho to the Rio Metlac, and in constructing "The Division of the Boot" (La Bota) in the center of which the Village of Maltrata is situated. He also served under Col. Talcott in Virginia.

Col. Talcott belonged to a period which furnished some of the most distinguished engineers to America—the pioneers who blazed the way. Among them were Messrs. Charles B. Fisk, Charles Ellett, and Benjamin H. Latrobe, and Colonels Claudius Crozet (a soldier of Napoleon's great army), C. F. M. Garnett, and Walter Gwynne—a

* Tucson, Ariz.

† New York City.

Mr. Douglas. galaxy the like of which is rarely found. In this company of great engineers, Col. Talcott, because of his high professional attainments and exalted character, measured up to the highest mark, and the writer recalls his service under him with the greatest pleasure, as it supplied a lesson which has never been forgotten.

Col. Talcott's sons, Charles, Richard, George, and T. M. R. (the latter a Colonel and Aid-de-camp to Gen. Robert E. Lee), all distinguished engineers, were the writer's personal friends and associates; he also recalls his service on this railroad with Messrs. William Cross Buchanan, Sebastian Wimmer, Richard Ingalls, and Dr. Manifred, with much pleasure.

Mr. Ingram. W. T. INGRAM,* ESQ. (by letter).—The writer has read Mr. Low's paper with deep interest. A great many of the data which he has gathered together, it is thought, have never before appeared in print; in fact, the present management of the railway was unaware that such a mass of information was still extant. The paper will prove a valuable addition to the Company's archives, and will be very interesting reading to those engineers who have been directly or indirectly connected with this, or, in fact, with any railway in Mexico.

This railway, opened for public service 43 years ago, is a lasting monument to the skill and energy displayed by the pioneers, and stands to-day a model of railway construction in Mexico.

Not until recent years has there been any serious attempt to improve the grades and alignment to meet the growing traffic, and it has been found that the most that can be done is to reduce the grade on the table-land, that is, from Esperanza to Mexico, from 1½ to 1%, and by building a high-level bridge over the Metlac River to save the expense of operating trains in and out of the ravine.

It is stated that the project of building a high-level bridge was abandoned—or, at least, very strongly influenced—on account of an earthquake which occurred while the project was under consideration; if this is the case, it was most unfortunate that the earthquake should have occurred at such an inopportune moment, for during the writer's long residence of nearly 28 years, spent wholly between the City of Mexico and Vera Cruz, he has never known, nor even heard from "the oldest inhabitant", of any earthquake which would have imperiled a high railway bridge, that is, of course, in the neighborhood of Orizaba. On the table-land he has experienced several severe disturbances, but these are reduced in intensity as the coast is approached; in fact, a severe earthquake in the City of Mexico is barely noticeable in Vera Cruz.

If it was unfortunate that the high-level bridge was not built in 1866; it is still more unfortunate that the Revolution, which commenced

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Mr. Ingram.

in 1910 and still continues, has deferred its construction indefinitely, due to the fact that, after long deliberation, it was considered that a bridge of such magnitude would be very easily destroyed, with, of course, disastrous consequences. Fortunately, the present Metlac Bridge and practically all others have escaped serious damage, but the danger has been ever present for several years past, and very great anxiety has been felt by all concerned; how much more so would this have been the case had the bridge been nearly 350 ft. high?

Some day, of course, the bridge will be built, as the development in and out of the *barranca* is perhaps the most dangerous part of the railway, and mile for mile the most expensive to operate. When this day arrives, it is to be hoped that the Mexican people will have developed into a peaceful and law-abiding nation.

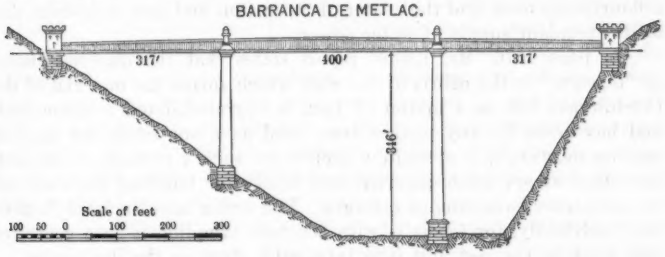


FIG. 11.

Mr. Lyons, in his report, condemned the route *via* Las Vigas and Jalapa, stating that the country was most unfavorable and that all cuttings would be in volcanic rock. His prognostication, however, has not been fulfilled, as the Interoceanic Railway, a narrow-gauge (36-in.) competing line with the Mexican Railway, follows the route which Mr. Lyons considered so unfavorable, and, moreover, the grades do not exceed 2½%, as compared with 4% (both uncompensated) of the Mexican Railway, the maximum curvature on both lines being practically the same, namely, 100 m.

The Interoceanic as it stands to-day is 50 km. longer than the Mexican Railway, but some 9 years ago a line was actually located from Las Vigas *via* Jalapa, to the foot of the mountain, using the same grade, namely 2½%, but compensated with maximum curvature of 200 m. The distance was shortened considerably, but this location involved the construction of an entirely new line for a length of some 125 km., consequently the project is hardly likely to be entertained favorably for some years to come. As a matter of fact, there is less rock on the Interoceanic route; the lava flow from the "Cofre de Perote", which has to be crossed, no doubt on first acquaintance looked very formidable, but its natural fall happens to fit the grade, and

Mr. Ingram.

the cuts and fills are relatively light. As the original scheme was to connect the City of Mexico with Vera Cruz, to a great extent irrespective of the intervening towns, except Puebla, which very rightly has been connected to the main line by a branch, the writer believes that, had the Jalapa route been studied as thoroughly as that of Orizaba, the former would have been selected, due to the more favorable grade.

On the other hand, however, the Orizaba route passes through towns and villages of greater importance and a much more cultivated and productive part of the country; consequently, on the whole, and in the light of after events, the selection of the route of the first railway in Mexico was a happy one. Jalapa is of no commercial importance, it being merely the seat of the State Government, whereas Orizaba is a flourishing town and the center of the cotton and jute industries, due to its abundant supply of water power.

On page 1576, Mr. Lyons' report states that the material known as "tepetate" is the débris of the rock which forms the material of the Cordilleras; but, as a matter of fact, it is consolidated volcanic mud, and has never by any chance been used as a substitute for sand in making mortar; it is absolutely useless for such a purpose, or, in fact, any other except when quarried into blocks for building purposes and for temporary ballasting of railways. The writer uses the word "temporary" advisedly, for tepetate, with constant tamping, is soon converted into mud in the wet and into intolerable dust in the dry season.

Mr. Perez Castro.

L. Perez Castro,* M. Am. Soc. C. E. (by letter).—This paper is very useful to those interested in the history of the leading railways of Mexico. The history of the grants and concessions, as well as of the progress of construction of the Mexican Railway, can be learned from the official reports of the Mexican Government, but very little has been published in reference to the engineering achievements of the pioneers on this wonderful railway. In fact, apart from the papers and reminiscences of the living men of that epoch, it is rather difficult to get extensive information about the matter.

At present, the reports of Capt. Talcott and his assistants have essentially an historical value, as the ideas now prevailing in reference to the location and construction of railways in Mexico are different from those of a quarter of a century or more ago, on account of the dissimilar conditions now obtaining. There are some engineers in Mexico, familiar with the country crossed by the Mexican Railway, who believe that its location could have been improved by following another line, but adhering to the same route south of the "Pico de Orizaba".

At the present time the volume of traffic carried by the railways of Mexico is far greater than that of former years, but, on the other

* City of Mexico, Mexico.

Mr. Perez Castro.

hand, the cost of fuel has increased considerably. A large proportion of the coal has to be imported. Recently, the use of Mexican fuel oil has resulted in great economy in the transportation expenses of these railways, but not enough to determine a change in the criterion relating to the maximum grades, for the price of fuel oil is steadily increasing.

The construction of the Interoceanic Railway, *via* Jalapa, is more in harmony with the actual requirements as to ruling grades, in the descent from the "Mesa Central" (Central Plateau) to the sea-level lands, as its heaviest grades are from 2.5 to 3%, instead of the 4.5% of the Mexican Railway (in both cases without compensation for curvature). The maximum curvature in each is about 12° (20-m. chord), or between 90 and 100 m. of radius. Wherefore, the Interoceanic Railway, in spite of the handicap of its present narrow gauge (0.914 m., or 3 ft.), has secured a greater freight traffic than the Mexican Railway, of standard gauge. It can be estimated that, before the outbreak of the political disturbances in Mexico, 60% of the freight from Vera Cruz to the table-lands was handled by the Interoceanic and the remainder by the Mexican Railway. The passenger movement has always been considerably superior on the latter railway, owing to the greater speed and comfort in making the trip, and also on account of its more attractive scenery.

With the facilities now available in the line of construction, and considering the necessities of an intensive traffic, it is admitted that the maximum grade suitable in the mountainous parts of Mexico, *viz.*, in climbing to the high lands, is 2% compensated for curvature, with maximum curves of 6° and, exceptionally, 8° (191 and 143 m. of radius, respectively, 20-m. chord). With this grade and 6° of sharp curves, a new line has been located between Las Vigas and Vera Cruz, on the Interoceanic Railway, and this will surely be constructed when the contemplated widening of the present narrow gauge takes place. The projected short line between Mexico City and Tampico—which is a pressing necessity—undoubtedly will be realized with such maxima of grade and curvature, notwithstanding the tremendous difficulties to be overcome in descending from the "Mesa Central", whichever route may be selected as suitable for the purpose. The different and numerous surveys have demonstrated that with 2.5% grades, the work would be somewhat simplified, although the prevailing tendency favors the maximum of 2 per cent.

With modern developments in electrification matters, and the use of hydro-electric power—which can be improved to a great extent in the Mexican mountains—the present limitation of grades can be economically increased; but, in the construction of new lines, the tendency at present is to exceed such limitations and use such lines exclusively for passenger and express purposes, like the Mexico-Puebla

Mr. Perez Castro.

line, passing between the snow-capped volcanoes, Popocatepetl and Ixtaccihuatl, and the Mexico-Toluca line, through the "Sierra de las Cruces", the construction of which was being carried out by the Mexico Tramway Company when it was delayed by the political troubles of the country. The maximum grade adopted on these lines is 6%, but on heavy-traffic lines already in existence, such as the Mexican Railway, surely in the near future, hydro-electric energy will be used, resulting in greater economy than that obtained with fuel oil as a substitute for coal.

On the lines in the "Mesa Central", the southerly part of which is essentially hilly, it is now estimated that 1% is the maximum economic grade, compensated, with 6° of greatest curvature, instead of the 1.5% grade largely used hitherto. The section of the Mexican Railway between Mexico City and Boca del Monte reaches in some stretches 1.5%, but surveys have been made with the object of reducing it to 1%, and it is possible that, when the conditions of the country are improved, this work will be started. On the Interoceanic Railway, between Mexico City and Las Vigas, similar surveys have been made for the purpose of making the necessary betterments in location when the road is widened to standard gauge. In the low country, lying between the foot of the mountains and the sea, the reasonable maximum grade is 0.5%, compensated, with maximum curvature of 2% (573-m. radius, 20-m. chord) and eventually 4° (287 m.). Thus has been located the coast line, pending construction, by the National Railways of Mexico, between Vera Cruz, Tampico, and Matamoros, the latter point on the United States border. In the surveys of the line from the Isthmus of Tehuantepec to the Yucatan Peninsula, grades of 1% have been proposed—and higher still in the neighborhood of Campeche—but this should not be approved as it would indicate a bad selection of the route, the land making it easy to keep the limit at 0.5 per cent.

Mr. Talcott.

T. M. R. TALCOTT,* ESQ. (by letter).—When Capt. Talcott assumed the duties of Chief Engineer of the Mexico and Pacific Railroad, in December, 1857, he was in his sixty-first year, and it was deemed desirable that one of his elder sons should accompany him as his personal assistant. For this the writer was to some extent qualified as he had had 18 months of previous training in that capacity on the Ohio and Mississippi Railroad, and had assisted his father in astronomical observations for the State of Virginia.

Some preliminary examinations of the ascent from Vera Cruz to the *cumbres* had been made by Mr. P. Almazan, a Mexican engineer, prior to the arrival of the expedition at Vera Cruz on January 4th, 1858. As Capt. Talcott was anxious to verify the barometrical eleva-

* Richmond, Va.

tions of some of the passes ascertained by Mr. Almazan, without waiting for the delayed transportation, he and the writer left Vera Cruz by diligence on the night of January 10th, and arrived at Orizaba the next afternoon. Mr. Talcott.

Having obtained saddle horses and a competent guide, the party started early on January 12th on the first of the many explorations on horseback in advance of the engineer parties, spending 5 days in the saddle, during which an examination was made of the route of the Diligence Road *via* Aculcingo as far west as Puente Colorado, a summit south of Aculcingo, which Mr. Almazan reported lower than the Aculcingo Pass, but which was found to be higher; the Canada de Regina Pass was also examined, and a route between Orizaba and Cordova which crossed the Barranca de Metlac north of the Diligence Road. The return to Vera Cruz was made by diligence on January 17th, and soon thereafter the engineer parties were fully equipped and in the field.

After a week spent at "Camp Escandon", near Vera Cruz, Capt. Talcott's next start was on horseback with the writer, and a Mexican *mozo* to care for the horses. The writer finds, from the only notebook which he has preserved from that period, that from January 25th to 31st, while on the way to Orizaba, he recorded astronomical and barometrical observations at Soledad, Chiquihuite Summit, and Potrero.

Early in February, Capt. Talcott established his headquarters at Orizaba, where he opened an office to which Mr. Coolidge and his assistants were moved from Camp Escandon; and thereafter hourly readings of the barometer and thermometer were recorded there, for comparison with contemporaneous observations by the surveying parties in order to determine elevations at controlling points along the Orizaba route then being surveyed. Astronomical observations were also made regularly for the rating of the several chronometers.

Capt. Talcott made a number of explorations both east and west of Orizaba during February, and the writer's notebook contains records of repeated observations at Maltrata, and at Aguila and Aculcingo Passes during that month.

Later, when there was fighting at Orizaba, Mr. Coolidge joined the "Liberal" party and was made prisoner by the "Conservadores", which for a time complicated the relations of the engineers with the military authorities of that party. Before that, when challenged by sentinels, the engineers were allowed to pass by saying that they were "American Engineers", but afterward they had to explain to officers at military posts that they were "Railroad Engineers" before being allowed to pass.

When there was no longer doubt of the practicability of the Maltrata route, Capt. Talcott extended his personal explorations west of the *cumbres* into the open country of the plains. On these reconnaissances each successive course by compass was recorded, and its length was

Mr. Talcott.

estimated by the time consumed in traversing it at a steady pace on horseback. In the middle of the day a halt was made in order to get the time by equal altitudes, and the approximate latitude of the halting place by circummeridian altitudes of the sun.

At night, both latitude and longitude were again determined by observations of the stars, thus fixing approximately the geodetic position of two points on each day's journey, from which the latitude and departure of each half day's journey was calculated and used in correcting the approximate scale of estimated distances. From these data the field parties were furnished with rough maps showing the routes traveled and such topographical features as had been sketched *en route*.

On May 25th, Capt. Talcott left Orizaba on horseback, accompanied by the writer and a Mexican *mozo*, and soon thereafter established his headquarters at Puebla. While there, on June 19th, the party had its first experience with an earthquake, but the shock was more severe in the City of Mexico.

On July 12th, according to the diary, the party left by private conveyance for Tlascala "to explore the route from Esperanza to Huamantla and Apam." While on this trip, the writer was left at Nopalucan, Capt. Talcott says, "to take charge of the line commenced by Lyon at Esperanza *via* Nopalucan, etc." This was the line which was finally adopted and built from the Boca del Monte to Apizaco. While the writer was engaged on this line, Capt. Talcott returned to Puebla, and, on July 20th, went thence *via* San Martin to Apam, from which point he made explorations "as far west as Saupauca [Soapayuca], in sight of Lake Texcoco and Otumba."

Starting again from Puebla on July 28th, he says he found the writer's party at Huamantla, and went thence, *via* San Andres to Orizaba, and thence returned to Puebla by August 4th. There the writer rejoined him and went with him to the City of Mexico on the 7th, and found that Don Manuel Escandon was in prison because he had refused to pay a forced loan of $400 000 demanded by the *de facto* government of Mexico. No one was allowed to visit him in prison, and after remaining in the city for a week, Capt. Talcott and the writer returned to Puebla without seeing him.

When the surveys were nearing completion, and the question of a general map was being considered, it occurred to the writer that it would be more complete and satisfactory if the Diligence Road from Puebla to the City of Mexico could be shown on it.

Time was short, and all the regular parties were otherwise engaged, so the writer offered to undertake the work and complete it in one week with such assistance as could be spared, including only Mexicans as rodmen and chainmen.

Mr. Talcott.

Half a day was lost in starting, by reason of an oversight in not getting a written permit from the Alcalde of the city, which led to the arrest of all in the party except the writer.

The first 20 miles to San Martin were not deemed essential, as this part was covered by information previously obtained, and as time was short the party hastened to that place, where the actual survey was commenced the next morning, starting with the elevation which had been ascertained by repeated observations of the barometer on previous trips by diligence to and from the City of Mexico.

From San Martin to Ayotla the distance was measured by chain, courses were taken by compass, elevations determined by level and rod, and as many as possible of the topographical features of the country between the Diligence Road and the line *via* Apam were fixed by triangulation.

On the morning of the sixth day, after passing over an elevation of about 10 200 ft. at Rio Frio, the party was on the shore of Lake Texcoco, the level of which above the sea had already been determined by the survey *via* Apam, and was surprised to find that the elevation of the lake differed only a few feet from that determined by the survey from Vera Cruz.

The dome of the Cathedral in Mexico being in plain view, its distance was determined by triangulation, and also the course of the causeway across Lake Texcoco, which, for the want of more time, was the best that could be done to connect the line with the survey *via* Apam.

After returning to the United States, in October, 1858, the writer remained with Capt. Talcott for several months, rendered him some assistance in the preparation of his report, and accompanied him to Arizona in 1860, during the interval in which he was waiting for his recall to Mexico.

In 1861, when Don Antonio Escandon sent a special messenger through the Federal lines with letters to Capt. Talcott asking his immediate return to Mexico, neither he nor the writer felt that they could leave the service of the State of Virginia in which they were then engaged on river, coast and harbor defenses; and, later, when Capt. Talcott did return to Mexico, the writer was in the Confederate Army, where he remained until April 9th, 1865, when Gen. R. E. Lee, on whose staff he had been serving, surrendered to Gen. U. S. Grant.

In the spring of 1866, when Capt. Talcott was in great need of engineers to replace those who had left, or were about to leave him, for service with the English contractors, the writer returned to Mexico, taking several assistants of his own selection, and succeeded Gen. W. H. Stevens, who had been in charge of construction on the Maltrata Incline as Division Engineer.

Mr. Talcott. During the months which followed, until the following February, when the French army was withdrawn, the chief occupation was measuring up, and preparing estimates for work done, which were the basis of a subsequent settlement with the English contractors.

In conclusion, the writer cannot refrain from saying that, of all the engineers who served with Capt. Talcott on the survey in 1858, and the work of construction between Vera Cruz and Paso del Macho in 1862 and 1863, Mr. M. E. Lyon, who met his death at the hands of Mexican bandits while in the discharge of his duty, was most conspicuously identified with the enterprise.

Mr. Taylor is mistaken in saying that Mr. Escandon had no interests at Jalapa, however, the writer believes that his personal interests were better served by the construction of the Orizaba line.

His arrangement with Capt. Talcott, made in New York with the approval of the representative of the Mexican Government, contemplated the survey of both the Jalapa and the Orizaba routes, and his engineer corps was organized by Capt. Talcott with that end in view.

It was only after Antonio Escandon arrived in Mexico, in April, 1858, and saw the difficulty in carrying out his original instructions, that further surveys on the Jalapa route were abandoned, and a large part of the engineer corps was disbanded.

When Capt. Talcott returned to Mexico, in 1862, the construction of the line *via* Orizaba was already in progress, and he was in no way responsible for that fact.

What part the French took in promoting the construction on the Orizaba route the writer does not know, but it was said that the French engineers made such elaborate surveys for the purpose of finding a better line than Capt. Talcott's in the direction of Orizaba, that the monkeys in the Chiquihuite Mountains caught the idea and imitated them.

Mr. Taylor may be correct in his opinion that the Interoceanic has by far the better natural route, but, so far as the writer knows, even at this late day, it is an open question, between the Orizaba and Jalapa route, as to which is the best for a railroad, all things considered.

Mr. Taylor. H. M. Taylor,* M. Am. Soc. C. E. (by letter).—The writer has had no connection with the Mexican Railway, but has made frequent trips over it while in charge of the construction and reconstruction of parts of the Vera Cruz and Pacific. The Mexican Railway, a typical English product, has the good features common to all the British built lines—good alignment, good roadbed, well-maintained, good stations, good section houses—and the fault of most of them,

* Morón, Camagüey, Cuba.

namely, the sacrifice of grade to alignment. Its curves were not compensated, neither are those of the British lines in Argentina, Uruguay, or Chili. In approaching divides, like that at Ometusco, between the Valley of Mexico and the Apam Valley, on the Gulf slope, the line makes straight for the hills, holding to the valley and taking a 1.50 grade in order to get over the divide, when, by a slight sacrifice of alignment, a 1% grade could have been had, using the slope which Nature provided; or westbound, an 0.50% grade, as has been built on the Interoceanic. This error was repeated at Guadalupe, at Paso del Macho, and at the Metlac Barranca. Considering its date of construction, however, and the condition of the Science of Engineering at the time, its location is notable. Mr. Taylor.

There were several revolutions between its commencement and its completion. Mr. Escandon had interests at Cordoba, and none at Jalapa, and naturally he constructed the line where his interests lay. Cordoba, to-day, due to its water-power, has far more freight to offer than Jalapa, and, as stated by Mr. Ingram, it is fortunate that the cost of operation on such grades was not then understood, or the Mexican Railway would have been built *via* Jalapa, and the two lines would have remained one. The Mexican made directly for Mexico and tapped Puebla *via* a branch, which was the sensible thing to do (notwithstanding Mr. Wellington). The percentage of business from Mexico to Vera Cruz was so much greater than that *via* Puebla that adding the cost of the longer haul *via* Puebla to the freight to and from Mexico would have penalized the public in the City of Mexico and north and west of that point beyond any reason.

The proof that Mr. Wellington was wrong in his theory is seen in the construction, at a much later date, of the line from San Lorenzo to Oriental by the Interoceanic.

It is not the writer's object, however, to discuss these matters, but to bring before engineers who are interested just what was made of the American-built, or rather located, line. In doing this he wishes to state that it has been always his understanding that Mr. Wellington had practically nothing to do with the actual location or construction of the line. The writer was not in Mexico at the time, but has known many Americans and other foreigners who were, some of whom were in position to know. Mr. Wellington was responsible for the route; not that the route had not been looked over before, for the Palmer-Sullivan concessionaires started a line from Vera Cruz to Mexico, built it to Antigua, about 40 km., did some grading between Perote and Limon, still to be seen, and some between Irolo and San Lorenzo, likewise still to be seen.

The writer never knew definitely what stopped this work, but he has been told that it was a complaint by the Mexican Railway of the infringement of its original and perpetual concession of a line between

Mr. Taylor.

the City of Mexico and Vera Cruz. The Interoceanic was formed of a number of separate concerns such as the "Ferrocarril de Irolo", extending from Mexico to Irolo and later to Iturbe; the Puebla to San Martin Tram line, from Puebla, 40 km. north through the Valle de Nativitas; and the Tram Company from Puebla to San Juan de los Llanos, *via* San Marcos and Virreyes. The latter was bought up and connected by short pieces of line, almost always badly located and badly constructed, because of an unfortunate contract with Don Delfin Sanchez, a promoter, a wonderfully able man in many ways, but who had no knowledge of railroad operation. Also, his contract was at a fixed price per kilometer, and, naturally, it was to his interest to fit the grade to the contours as nearly as could be done.

Between Virreyes and Rubin, the Interoceanic was constructed through a wide valley, and a tangent for the whole distance was an easy proposition. One can see from Limon to Oriental with absolutely nothing in the way of a water-grade tangent. Through this the line zigzagged, up over a hill and down again, got a little nearer Tepeyahualco, a small Indian village, on basalt rocks, not worth a car of freight per week twenty years after the line was built.

From Limon to Rubin the line was fairly good, with an ascending grade of 0.73, and 2° curves. The same extended east nearly to Rio Frio; here, rough country was encountered, and a succession of grades up and down, 2.50% in both directions, with maximum curves at the top and bottom of each grade. The line ascended 38 m. to La Cima, and ran down again to Las Vigas, the natural summit of the country, the distance being 15 km. A revision of the line, built while the writer was in charge, reduced the grade to a supported 0.35 and a maximum curve of 3 degrees. The line was shortened 1¼ km., and, in this short distance, more than 1 100° of curvature were eliminated. The quantities of the revision were nearly three times those of the original line, but the cost was very nearly the same, due to the fact that nearly all the original line was in rock, and the revision was nearly all in earth.

In operation, the revision cut down the helper-engine distance 28 km.; trains in both directions were helped, and the estimate showed more than 20% per annum on the cost of making the change.

The line from Metepec to San Lorenzo was worked out on the basis of ton-kilometer cost, and when revision was made, the net returns to the Company were estimated as being more than 15 per cent. An examination of the returns of the Company after these revisions were made showed that the estimates were conservative.

Below Las Vigas the line descends on a 2.50% grade to Banderilla; then there is a light grade crossing over into another drainage, with a 2.50% adverse grade to above Bruno, descending again on a 2.50% grade to Jalapa. This work of revision was heavy, and business did

not yet justify the change there. It was close at hand in 1911, however, together with that to Las Vigas. The writer disagrees with Mr. Ingram in believing that the Company, having made two investments which proved so profitable, will not make another along the same lines when peace happily returns to Mexico. Mr. Taylor.

Below Jalapa, the revision showed that the old Camino National, along which the Mexican Tramway ran in Mr. Wellington's time, was the proper route, a 2% line with a maximum curve of 6° being found and compensated. The tonnage was calculated, and the quantity required to make the investment in the change of line profitable was determined. That, too, was not so far away when the Madero Revolution caused postponement of the question of investments in all Mexican projects excepting oil lands.

Unlike Mr. Ingram, and others of the Mexican Railway, the writer is not proud of the location of the Interoceanic. It is a fine property, and it is to be regretted that its location renders it inferior to the Mexican line as a transportation machine, when it could have been superior, as no doubt it will be when peace comes, prosperity returns, and it is made of standard gauge.

Emile Low,* M. Am. Soc. C. E. (by letter).—On March 4th, 1517, Francisco Hernandez de Cordoba discovered the coast of Yucatan, and in the following year Juan de Grijalva landed on the island now occupied by the castle of San Juan de Ulua, off Vera Cruz. On April 21st, 1519, Hernando Cortez landed on the site of the present City of Vera Cruz. On August 16th, 1519, Cortez and his bold Conquistadores left Cempoalla, the capital of the Totonac Indians, on his famous march to the Aztec capital, Tenochtitlan, the site of the present City of Mexico. His force consisted of 400 Spanish soldiers, 15 horses, with 7 pieces of artillery, augmented by several thousand Totonac Indians. Mr. Low.

The line of march passed through the Indian village of Jalapa, thence to the north of the volcano, Cofre de Perote, reaching the table-land, across this to the Indian town of Tlaxcala, where a battle was fought with the natives, in which the Spaniards were victorious. From this place the march continued to Cholula, where another battle was fought between the Conquistadores and the Cholulans, in which several thousand of the latter were slain. The ascent of the mountain then began, Cortez crossing the saddle between the volcanoes of Popocatepetl and Iztaccihuatl, descending to the town of Amecameca, and then across the valley to the capital of Montezuma which was entered November 8th, 1519, less than 3 months after leaving the shores of the Gulf of Mexico.

It is hard to believe that the route traveled was anything more than a trail, perhaps the only one between the capital and the Gulf of

* Buffalo, N. Y.

Mr. Low. Mexico, a foot-path only. There were no horses in the country, and wheeled vehicles were unknown. Horses were brought to Mexico by the Spaniards, and during the two centuries which succeeded the Conquest the journey between the coast and the interior was made on horse, or mule back, or on foot. At the beginning of the Nineteenth Century, litters (*literas*) were used between Vera Cruz and Jalapa, and a line of coaches ran thence to the capital. In 1833 the first stage line was established between Mexico and Jalapa, and was extended later to the coast.

As streams flowed in an almost direct line from the top of the table-land to the coast, making the shortest descent, this was the reason for the first line of communication chosen. If there was another one, or more, the fact was not known at the time. Later, another trail was developed to the south of Orizaba, and still later, at about the end of the Eighteenth Century, they were transformed into wagon roads, with the dignified name of *camino real,* the King's highway.

On page 1546 these two highways between Vera Cruz and the City of Mexico are mentioned. The northern, passing to the north of Cofre de Perote, built at the expense of the Consulate of Mexico and constructed with Roman splendor, to serve as a passage for all the traffic carried on by New Spain at one period, was abandoned shortly after the Independence (1823). Its towns, which were maintained by a fictitious commercial movement, were depopulated little by little, and its fields were deserted.

The National or King's Bridge—Puente del Rey, as it was called during the colonial period—56 km. out from Vera Cruz, was constructed toward the end of the Eighteenth Century by Gen. Don Manuel Rincon. In its material aspect it had the appearance of a Roman viaduct, and the structure was executed not only scientifically, but with splendor, as it changes its direction to follow the course of the road.

When Capt. Talcott landed in Vera Cruz, in January, 1858, he found the conditions as stated. From the examinations made by him, he decided that the proper line for a railroad was *via* Orizaba, and his surveys were all based on this decision.

Subsequent events absolutely justified this action, and it would have been nothing less than a national calamity for Mexico had the Jalapa route been chosen at the time.

About 1873, Don Ramon Zangroniz, of Vera Cruz, obtained a concession for a tramway from Vera Cruz to Jalapa, to be operated by animal power. He succeeded in building only a short section of it, when, in 1874, the concession was transferred to the Mexican Railway Company, Limited, which continued and completed the construction in May, 1875.

Mr. Low.

This horse tramway actually began at the Tejeria station, 15 km. out from Vera Cruz on the Mexican Railway, and then continued in a straight line to San Juan, where the main highway (which passed north from Vera Cruz) was intersected and generally followed to Jalapa, passing through the towns and villages of Zopilote, Tierra Colorada, Paso de Ovejas, Puente Nacional, Rinconada, Plan del Rio, Cerro Gordo, and Dos Rios, the distance between Vera Cruz and Jalapa being 114 km. (71 miles), with a rise of 1 361 m. (4 465 ft.), of which 1 100 m. (3 608 ft.) are made between Rinconada and Jalapa, a distance of 50 km. (31 miles), an average grade of 2.2%, although it is stated that 10% grades abound, which statement needs verification.

This tramway, extending for more than 70 miles, was the longest horse railway in the world.

The late A. M. Wellington, M. Am. Soc. C. E., landed in Vera Cruz in March, 1881, and it may not be amiss to state here that the writer and his father, Sigismund Low, were fellow-travelers on the steamer from New York, having secured service with the Mexican Central Railway, then under construction between the City of Mexico and El Paso del Norte (now Ciudad Juárez), while Wellington and his principal assistant, the late H. H. Filley, M. Am. Soc. C. E., were employees of the Mexican National Railroad, a narrow-gauge line building between the City of Mexico and Laredo on the Rio Grande. As stated by Wellington,* his first assignment was to make a reconnaissance for a railroad line from Vera Cruz to Mexico, *via* Jalapa "sufficient to determine the general possibilities of the route" and on which a corps of engineers was already engaged. The reconnaissance made by Wellington occupied only 3 days, and the resulting report was "that a 2 per cent. grade (uncompensated) was practicable". Such a conclusion was easily arrived at, as all the necessary data were available. The natural topography of the country was such that a 2% grade (uncompensated) just fitted it, although it was necessary at a number of points to use an excessive development of line in order to maintain this low grade. It may be pertinent to ask here why Wellington called this line the "American Line". He, like many other engineers not familiar with railroad building in Mexico, labored under the belief that the Mexican Railway (Vera Cruz-Mexico) was surveyed, located, and constructed by English engineers, when the very opposite was true, that honor belonging to an American engineer, Capt. Andrew Talcott, whose name and fame is indelibly connected with it.

Some time after Wellington's reconnaissance, further surveys were made, which showed a rise of 7 323 ft. in 72.64 miles. Wellington then

* "The American Line from Vera Cruz to the City of Mexico, via Jalapa, with Notes on the Best Methods of Surmounting High Elevations by Rail," *Transactions*, Am. Soc. C. E., Vol. XV, p. 791.

Mr. Low. made a comparison of his line with that of the Mexican Railway, and, regarding their respective grades, says:

"Continuous 2 per cent. (uncompensated) against a broken 4 per cent. (uncompensated); including the effect of curvature or of compensation therefor, 2.6 per cent. against 6 per cent."

It is hard to understand what method of computation was used in arriving at the comparative rates of grades, after allowing for compensation. Wellington allowed for his line, in which 20° curves occurred, a rate of 0.03 ft. per degree, which would give 0.6 ft., or an equivalent 2.6% grade. For the Mexican Railway, he assumed a rate of 0.11% which would give 2.0 ft., or an equivalent 6.0% grade, certainly a most extraordinary method of calculation. With the same rate of curve compensation, the equivalent grade would have been, on the Mexican Railway, 4.5 per cent.

After a short connection with the Mexican National Railroad, Wellington returned to the United States, but soon after entered the service of the Mexican Central Railway, and, during his connection with this railway, 1881-83, he was an ardent advocate of 2% grades, under all conditions, very high curve compensation, and momentum grades, which requirements did not always fit every line, as was shown in the case of the Guanajuato Branch-Siloa-Marfil, in which the writer, under his father, was in charge of location and construction.

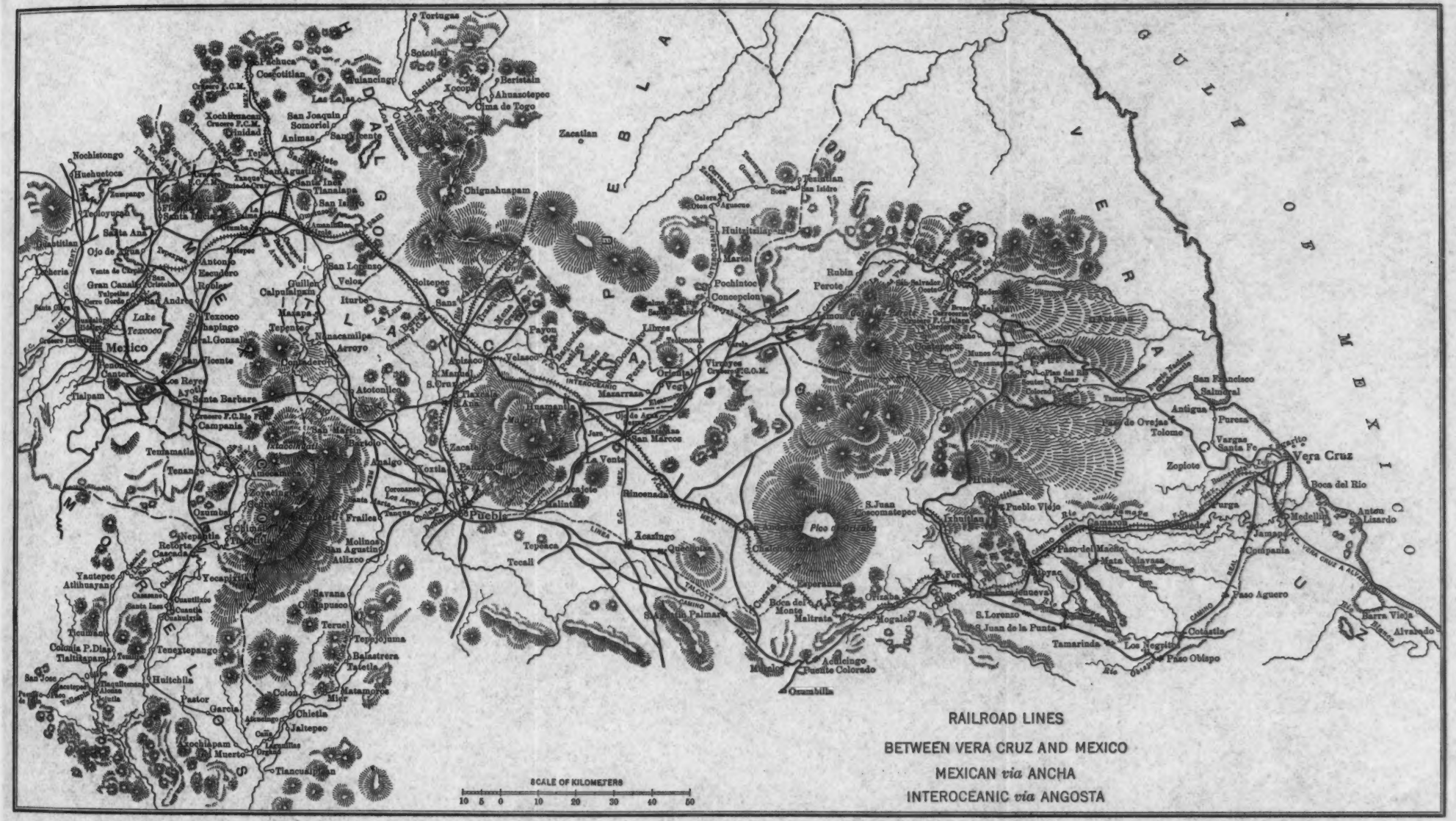
The original survey for this branch line was from a point between Irapuato and Siloa, following the valley of the Guanajuato River to the city of the same name. This would have placed the junction in an undeveloped plain, and Siloa was finally selected. To reach Guanajuato from this city it was necessary either to tunnel a high foot-hill or go over the top. A 3% grade just fitted the topography, and the line was thus located. Wellington ordered a 2% grade with high curve compensation, which, starting with a shallow cut at the summit, brought the line on reaching the main valley "high up in the air", necessitating much development to get down, and almost doubling the distance. It is needless to say that this low grade line was abandoned and the 3% line built.

The present Mexican Railway was incorporated on August 20th, 1864, as the Imperial Mexican Railway Company, Limited, under the government of Emperor Maximilian, being granted a subvention (concession or subsidy) of $560 000 a year, of which $420 000 was the property of the company for 25 years dating from November 11th, 1868.

The Mexican Government also agreed not to subsidize any other railroad between Vera Cruz and the City of Mexico for 65 years from November, 1868.

The railroad *via* Jalapa, 3-ft. or narrow gauge, was subsequently built under the name of the Interoceanic Railway, a link in the present National Railways of Mexico.

PLATE LVIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXX, No. 1371.
LOW ON
MEXICO AND PACIFIC RAILROAD.


Mexico
Puebla
Vera Cruz
Orizaba
Pico de Orizaba
Zacatlan
Chignahuapam
Huitzilapan
Pochintoc
Concepcion
Rubia
Perote
Jalapa
Antigua
Puente Colorado
Acultzingo
Maltrata
Boca del Monte
Paso del Macho
Paso Agüero
Cotastla
Tamarindas
Los Negritos
Paso Obispo
Barra Vieja
Alvarado
Boca del Rio
Medellin
Jamapa
Compania
Zopiote
Tolome
Puente Nacional
Vargas
Santa Fe
Puebla Viejo
Ixhuacan
Huatusco
Coscomatepec
S. Juan
Rincon Grande
Acazingo
Quecholac
Tepeaca
Tecali
Tlalixtac
Amozoc
Nopalucan
Teziutlan
Libres
Oriental
Virreyes
Perote
Xalapa
Banderilla
Coatepec
Cordoba
Fortin
Mirador
Rio Blanco
Nogales
Cuichapa
Mixtla
Tlacotepec
Tehuacan
Esperanza
Tepeyahualco
Tepeojuma
Teteles
Balastrera
Matamoros
Miahuatlan
Chietla
Atencingo
Jaltepec
Izucar de Matamoros
Tianguisillan
Tepexoxuca
Tenextepango
Tlaltizapan
Yautepec
Atlihuayan
Cuautla
Cuautlixco
Yecapixtla
Tetela del Volcan
Ocuituco
Tetela
Hueyapan
Tochimilco
Atlixco
Molinos
San Agustin
Tepapulco
Otumba
Apan
Calpulalpam
Texcoco
Lake
Tepexpan
Acolman
Otumba
Ometusco
Ayotla
Tlalpam
Santa Barbara
Tenamatla
Tenango
Ozumba
Amecameca
Ayapango
Chalco
Nexpantla
Rio Frio
Cascada
Yecapixtla
Cuautla
Cuautlixco
Santa Inez
Cuautlalpan
Tlaltizapan
Tenextepango
Huitchila
Pastor
Garcia
Colon
Matamoros
Miahuatlan
Chietla
Jaltepec
Izucar de Matamoros
Tianguisillan
Tepexoxuca
Tehuacan
Esperanza
Tepeyahualco
Tepeojuma
Teteles
Balastrera
Nopalucan
Amozoc
Tlalixtac
Tecali
Tepeaca
Quecholac
Acazingo
Rincon Grande
Coscomatepec
S. Juan
Huatusco
Ixhuacan
Pueblo Viejo
Boca del Rio
Vera Cruz
Medellin
Jamapa
Compania
Zopiote
Tolome
Puente Nacional
Vargas
Santa Fe
Antigua
Puente
San Francisco
Salmoral
Banderilla
Coatepec
Cordoba
Fortin
Mirador
Rio Blanco
Nogales
Cuichapa
Mixtla
Tlacotepec
Paso Agüero
Cotastla
Tamarindas
Los Negritos
Paso Obispo
Barra Vieja
Alvarado
Boca del Monte
Maltrata
Puente Colorado
Acultzingo
Orizaba
Pico de Orizaba
Zacatlan
Chignahuapam
Huitzilapan
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Perote
Jalapa
Antigua
Puente Nacional
Vera Cruz
Boca del Rio
Medellin
Jamapa
Compania
Zopiote
Tolome

Mr. Low.

It was chartered on April 30th, 1888, and purchased all the concessions granted by the Mexican Government for a railroad from Vera Cruz to Amacusac (near Puente de Ixtla) *via* Puebla and the City of Mexico. The main line *via* Puebla was opened from Mexico to Vera Cruz on April 1st, 1891, the distance being 339 miles.

Above and below Jalapa there are 3% grades. Later, about 1905, a cut-off was built between San Lorenzo and Oriental, 72 miles, which reduced the distance to 293 miles between Vera Cruz and the City of Mexico, as compared with 264 miles *via* the Mexican Railway.

The map, Plate LVIII, shows the lines of the Mexican and Interoceanic Railways between Vera Cruz and the City of Mexico.

The especial feature shown on this map is the more favorable alignment of the Mexican Railway, the sinuosities of the Jalapa line being very pronounced.

The highest elevation of the Maltrata grade on the Mexican Railway is between Boca del Monte and Esperanza, it being 2 470 m. (8 104 ft.) (although there is a slightly higher elevation, just east of San Andres, 2 485 m. (8 153 ft.) and on the Interoceanic at La Cima the elevation is 2 465 m. (8 087 ft.). There are descents on the former line aggregating about 130 m. (426.5 ft.), probably somewhat less on the latter line, so that the sums of the ascents and descents are a little in favor of the Jalapa line.

Regarding the highest elevation between Vera Cruz and the City of Mexico, Hacienda de Acocotla, between Apizaco and Huamantla, on the Mexican Railway is 2 537 m. (8 323 ft.) and, on the Interoceanic, Mena is 2 568 m. (8 425 ft.).

For a number of years, and likely now, all competitive traffic, both import and national, has been pooled by the two competing lines.

In reference to the report of Mr. Pascual Almazan on a line *via* Pueblo Viejo, Octitlan, Ixhuatlan, and San Juan Coscomatepec, it will be noted on the map, Plate LVIII, that the Mexican Railway has since built a railroad from Cordoba to the latter town. Thus, it will be seen that many of the railroad lines follow the old trails, which proves that to some extent the Indian was the pioneer engineer.

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Paper No. 1372

TEMPERATURE STRESSES IN A SERIES OF SPANS

By Tresham D. Gregg, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Charles W. Martin and Tresham D. Gregg.

Synopsis.

The general problem of the temperature stresses in a series of spans arose while the writer was preparing plans for a viaduct at Little Rock, Ark., to carry a street over the tracks of the Rock Island, and the St. Louis, Iron Mountain, and Southern Railroads. The type approved by the City and the Railroad Companies was a series of six simple spans composed of girders decked with concrete, and supported on solid concrete piers.

The proposal to build the three west spans of this viaduct integral with the piers and abutment, brought up the consideration of the temperature stresses.

This paper is a general solution of this problem by the method of least work and the Castigliano theorem for one, two, and three spans of variable stiffness and inelastic abutments, and an application of the general solution to this particular case.

A convenient method for the logarithmic computation of stresses is also given, and a brief discussion of the results of the investigation, with tables of constants and of computed stresses.

The writer hopes that the problem set forth in this paper may be of interest.

FIG. 1.—WEST THIRD STREET VIADUCT, LITTLE ROCK, ARK.

FIG. 2.—WEST THIRD STREET VIADUCT, LITTLE ROCK, ARK.

In the course of his work as Bridge Designer in the Bridge Engineer's office of the Chicago, Rock Island and Pacific Railway, the writer was required to prepare detailed plans for a viaduct at Little Rock, Ark., to carry West Third Street across the tracks of the Rock Island, and the St. Louis, Iron Mountain, and Southern Railroads. The general outline of the bridge and the approximate length and character of the spans had been previously agreed upon by the two railroads concerned and the city authorities.

The type originally proposed by the City was a series of reinforced concrete arches. That finally agreed upon, however, was a series of simple spans, the outer girders of which had somewhat the appearance of arches (Figs. 1 and 2). Each of the three east spans is composed of eight steel girders covered and decked with reinforced concrete. Each of the three west spans is composed of eight reinforced concrete girders with integral reinforced concrete deck. All spans are supported by solid concrete piers, 3 ft. wide at the coping, all of which are on a more or less pronounced skew.

The problem arose from the proposal to build the three west spans integral with the piers and abutment, that is, without provision for changes in length due to temperature and shrinkage.

It is evident that such piers offer resistance to the free expansion or contraction of the spans which they support, and hence must introduce corresponding stresses. Are these stresses negligible? If they are not, and expansion must be provided for, how many joints are necessary, and where should they be placed?

It will be seen from Fig. 3 that, although the piers are similar in design, they have different angles of skew, and therefore different degrees of stiffness in the line of the viaduct. The spans under consideration are of approximately constant section, except as to the quantity of steel reinforcement. The outer or sidewalk girders are continuous over the two intermediate piers. The other six are not.

Fig. 3 shows the average span lengths and the angles of skew of the piers. The areas of steel and concrete in the several spans, and the respective average moments of inertia of the piers about axes perpendicular to the axis of the bridge are given in Table 2. The heights of the piers in Fig. 1 are given from the base to the neutral plane of the spans, in each case. Although they vary somewhat, they are assumed to be equal, for simplicity in the calculations.

First, consider the simplest case—that of a single span held between solid, inelastic abutments, by frictionless pins axially placed, as shown in Fig. 4.

Case I.—One span on fixed, inelastic support. (Fig. 4.)

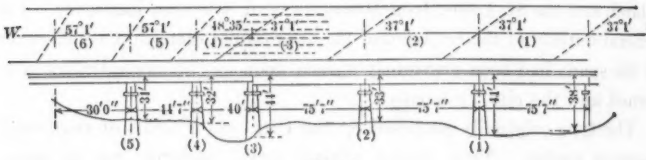


FIG. 3.

Let t equal the change of temperature, in degrees Fahrenheit;

e " " coefficient of expansion, in inches per inch per degree Fahrenheit;

E " " modulus of elasticity, in pounds per square inch;

A " " area of cross-section of span, in square inches;

l " " length of span, from center to center of piers, in inches.

Let T equal the total axial stress due to a temperature change of t degrees;

D " " change in length of span, if allowed to expand freely, $= t\ e\ l$.

Then $$T = \frac{D\ A\ E}{l}.$$

Let $$\frac{l}{A\ E} = C.$$

Substituting, we have: $$T = \frac{D}{C}.$$

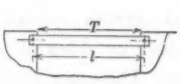


FIG. 4.

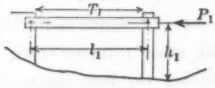


FIG. 5.

This must be added algebraically to the bending stresses in order to obtain the maximum fiber stress.

Case II.—One span fixed at both ends, one resting on an inelastic abutment, and the other on an elastic pier. (Fig. 5.)

Let the height of pier be h_1 inches; its moment of inertia, I_1; and its modulus of elasticity, E.

Suppose a force, P_1, to be applied axially with the span at the top of the pier. This force produces an axial stress, T_1, in the span, and bending stresses in the pier.

The deformation in the span due to the force, P_1, $= \frac{T_1 l_1}{A E}$, and the work done $= \frac{T_1^2 l_1}{2 A E}$.

The deflection of the pier due to the load,

$$T_1 - P_1 = (T_1 - P_1) \frac{h_1^3}{3 E I_1},$$

and the work done $= (T_1 - P_1)^2 \frac{h_1^3}{6 E I_1}$.

The total work done $= U = \frac{T_1^2 l_1}{2 A E} + \frac{(T_1 - P_1)^2 h_1^3}{6 E I_1}$.

Differentiating with respect to T_1:

$$\frac{d U}{d T_1} = \frac{T_1 l_1}{A E} + (T_1 - P_1) \frac{h_1^3}{3 E I_1} \quad \ldots\ldots\ldots\ldots\ldots (1)$$

Equating this to zero, by the principle of least work, T is found in terms of the force P_1.

If P_1 = zero, but the span changes in length, due to some other cause, such as a rise or fall in temperature, then, by Castigliano's theorem, the foregoing expression may be equated to the increment of length which the span would have if it were free from the restraint of the pier and abutment, that is,

$$\frac{T_1 l_1}{A E} + \frac{T_1 h_1^3}{3 E I_2} = D_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

As before, let $\frac{l_1}{A E} = C_1$, and let $\frac{h_1^3}{3 E I_1} = B_1$.

Substitute in Equation (2) and solve.

$$T_1 = \frac{D_1}{B_1 + C_1} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Case III.—Single span, both supports elastic. (Fig. 6.)

The axial deformation of the span $= \frac{T_2 l_2}{A E}$, and the work of deformation $= \frac{T_2^2 l_2}{2 A E}$.

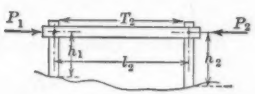


FIG. 6.

The deflection of Pier 1 $= (P_1 - T_2) \frac{h_1^3}{3 E I_1}$, and the work of deflection $= (P_1 - T_2)^2 \frac{h_1^3}{6 E I_1}$.

The deflection of Pier 2 $= (T_2 - P_2) \frac{h_2^3}{3 E I_2}$, and the work $= (T_2 - P_2)^2 \frac{h_1^3}{6 E I_1}$.

The total internal work:

$$U = (P_1 - T_2)^2 \frac{h_1^3}{6 E I_1} + \frac{T_2^2 l_2}{2 A E} + (T_2 - P_2)^2 \frac{h_2^3}{6 E I_2}.$$

Differentiating with respect to T:

$$\frac{d U}{d T_2} = (- P_1 + T_2) \frac{h_1^3}{3 E I_1} + \frac{T_1 l_2}{A E} + (T_2 - P_2) \frac{h_2^3}{3 E I_2}.$$

Let P_1 and $P_2 = 0$, and, as before, placing the resulting expression equal to the unrestrained elongation, D_2,

$$\frac{T_2 h_1^3}{3 E I_1} + \frac{T_2 l_2}{A E} + \frac{T_2 h_2^3}{3 E I_2} = D_2 \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Solving for T_2 and making the same substitutions as before:

$$T_2 = \frac{D_2}{B_1 + B_2 + C_2} \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Case IV.—Two spans on one inelastic abutment, and two elastic piers. (Fig. 7.)

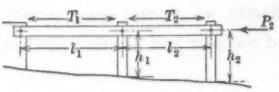


FIG. 7.

In a similar manner, the work of deformation due to the force $P_2 = U$.

$$U = \frac{T_1^2 l_1}{2 A E} + (T_1 - T_2)^2 \frac{h_1^3}{6 E I_1} + \frac{T_2^2 l_2}{2 A E} + (T_2 - P_2)^2 \frac{h_2^3}{6 E I_2}.$$

Taking the derivatives of U with respect to T_1 and T_2 successively:

$$\frac{d U}{d T_1} = \frac{T_1 l_1}{A E} + (T_1 - T_2) \frac{h_1^3}{3 E I_1} = D_1 \ldots\ldots\ldots\ldots (6)$$

$$\frac{d U}{d T_2} = (- T_1 + T_2) \frac{h_1^3}{3 E I_1} + \frac{T_2 l_2}{A E} + (T_2 - P_2) \frac{h_2^3}{3 E I_2} = D_2 . (7)$$

As before, making the B and C substitutions, we have:

$$(B_1 + C_1) T_1 - B_1 T_2 = D_1 \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

$$- B_1 T_1 + (B_1 + B_2 + C_2) T_2 = D_2 \ldots\ldots\ldots\ldots\ldots (9)$$

Solving simultaneously for T_1 and T_2:

$$T_1 = \frac{(B + B_2 + C_2) D_1 + B D_2}{(B + C)(B_1 + B_2 + C_2) - B_1^2} \quad \ldots\ldots (10)$$

$$T_2 = \frac{B_1 D_1 + (B_1 + C_1) D_2}{(B_1 + C_1)(B_1 + B_2 + C_2) - B_1^2} \quad \ldots\ldots (11)$$

If $B_1 = B_2$: $D_1 = D_2$: $C_1 = C_2$,

$$T_1 = \frac{(3B + C)}{(B + C)(2B + C) - B_2}$$

$$T_2 = \frac{(2B + C)}{(B + C)(2B + C) - B_2} \quad \ldots\ldots (11')$$

And $$T_1 = \frac{3B + C}{2B + C} T_2 \quad \ldots\ldots (10')$$

Case V.—Two spans; all supports elastic. (Fig. 8.)

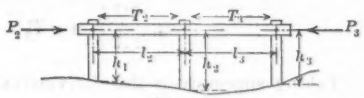


FIG. 8.

The work of deformation due to the forces, P_2 and P_3, $= U$.

$$U = (P_2 - T_2)^2 \frac{h_1^3}{6 E I_1} + \frac{T_2^2 l_2}{2 A E} + (T_2 - T_3)^2 \frac{h_2^3}{6 E I_2} + \frac{T_3^2 l_3}{2 A E} + (T_3 - P_3)^2 \frac{h_3^3}{6 E I_3}.$$

Taking successively the derivatives of U with respect to T_2 and T_3, and placing equal respectively to D_2 and D_3:

$$\frac{dU}{dT_2} = (-P_2 + T_2) \frac{h_1^3}{3 E I_1} + \frac{T_2 l_2}{A E} + (T_2 - T_3) \frac{h_2^3}{3 E I_2} = D_2 \ldots (12)$$

$$\frac{dU}{dT_3} = (-T_2 + T_3) \frac{h_2^3}{3 E I_2} + \frac{T_3 l_3}{A E} + (T_3 - P_3) \frac{h_3^3}{3 E I_3} = D_3 \ldots (13)$$

Letting P_2 and $P_3 = 0$, and making the B and C substitutions, we have:

$$(B_1 + B_2 + C_2) T_2 - B_2 T_3 = D_2 \quad \ldots\ldots (14)$$

$$-B_2 T_2 + (B_2 + B_3 + C_3) T_3 = D_3 \quad \ldots\ldots (15)$$

Solving simultaneously for T_2 and T_3, we have:

$$T_2 = \frac{(B_2 + B_3 + C_3) D_2 + B_2 D_3}{(B_1 + B_2 + C_2)(B_2 + B_3 + C_3) - B_2^2} \quad \ldots\ldots (16)$$

$$T_3 = \frac{B_2 D_2 + (B_1 + B_2 + C_2) D_3}{(B_1 + B_2 + C_2)(B_2 + B_3 + C_3) - B_2^2} \quad \ldots\ldots (17)$$

If

$$B_1 = B_2 = B_3 = B$$
$$C_2 = C_3 = C$$
$$D_2 = D_3 = D$$

$$T = \frac{D}{C + B} = T \; (= \text{Case II}).$$

Case VI.—Three spans; one inelastic abutment, and three elastic piers. (Fig. 9.)

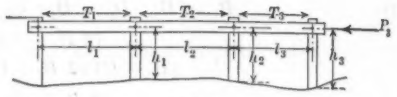


FIG. 9.

The work of deformation due to the external force, P, $= U$.

$$U = \frac{T_1^2 l_2}{2 A E} + (T_1 - T_2)^2 \frac{h_1^3}{6 E I_1} + \frac{T_2^2 l_2}{2 A E} + (T_2 - T_3)^2 \frac{h_2^3}{6 E I_2}$$
$$+ \frac{T_3^3 l_3}{2 A E} + (T_3 - P_3)^2 \frac{h_2^3}{6 E I_2}.$$

Taking successively the derivatives of U with respect to T_1, T_2, and T_3, we have:

$$\frac{d U}{d T_1} = \frac{T_1 l_1}{A E} + (T_1 - T_2) \frac{h_1^3}{3 E I_1} = D_1 \ldots\ldots\ldots (18)$$

$$\frac{d U}{d T_2} = (- T_1 + T_2) \frac{h_1^3}{3 E I_1} + \frac{T_2 l_2}{A E} + (T_2 - T_3) \frac{h_2^3}{3 E I_2} = D_2 \ldots (19)$$

$$\frac{d U}{d T_3} = (- T_2 + T_3) \frac{h_2^3}{3 E I_2} + \frac{T_3 l_3}{A E} + (T_3 - P_3) \frac{h_3^3}{3 E I_3} = D_3 \ldots (20)$$

Letting $P_3 = 0$, and making the B and C substitutions, we have:

$$(B_1 + C_1) T_1 - B_1 T_2 = D_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$
$$- B_1 T_1 + (B_1 + B_2 + C_2) T_2 - B_2 T_3 = D_2 \ldots\ldots\ldots (22)$$
$$- B_2 T_2 + (B_2 + B_3 + C_3) T_3 = D_3 \ldots\ldots\ldots\ldots\ldots\ldots (23)$$

Solving simultaneously for T_1, T_2, and T_3, we have:

$$T_1 = \frac{D_1 + B_1 T_2}{B_1 + C_1} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (24)$$

$$T_2 = \frac{B_1 (B_2 + B_3 + C_3) D_1 + (B_1 + C_1) (B_2 + B_3 + C_3) D_2}{(B_1 + C_1)(B_1 + B_2 + C_2)(B_2 + B_3 + C_3) - (B_1 + C_1) B_2^2}$$
$$\frac{+ (B_1 + C_1) B_2 D_3}{- B_1^2 (B_2 + B_3 + C_3)} \ldots (25)$$

$$T_3 = \frac{D_3 + B_2 T_2}{B_2 + B_3 + C_3} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (26)$$

If $$D_1 = D_2 = D_3 = D,$$

$$B_1 = B_2 = B_3 = B,$$

and $$C_1 = C_2 = C_3 = C,$$

then $$T_1 = \frac{D + B\,T_2}{B + C} \qquad (24')$$

$$T_2 = \frac{D\,(B\,(3\,B + C) + (B + C)\,(2\,B + C))}{(B + C)\,(2\,B + C) - B\,(3\,B + C)} \qquad (25')$$

$$T_3 = \frac{D + B\,T_2}{2\,B + C} \qquad (26')$$

And $$T_1 = \frac{(2\,B + C)\,T_3}{B + C} \qquad (26'')$$

Case VII.—Three spans; all supports elastic. (Fig. 10.)

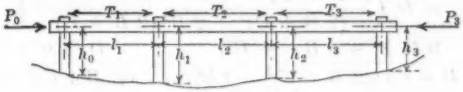


FIG. 10.

The work of deformation of the forces, P_0 and P_3, is equal to U.

$$U = (P_0 - T_1)^2 \frac{h_0^3}{6\,E\,I_0} + \frac{T_1^2\,l_1}{2\,A\,E} + (T_1 - T_2)^2 \frac{h_1^3}{6\,E\,I_1} + \frac{T_2^2\,l_2}{2\,A\,E}$$

$$+ (T_2 - T_3)^2 \frac{h_2^3}{6\,E\,I_2} + \frac{T_3^2\,l_3}{2\,A\,E} + (T_3 - P_3)^2 \frac{h_3^3}{6\,E\,I_3},$$

whence, $$\frac{d\,U}{d\,T_1} = (-P_0 + T_1) \frac{h_0^3}{3\,E\,I_0} + \frac{T_1\,l_1}{A\,E} + (T_1 - T_2) \frac{h_2^3}{3\,E\,I_2}$$

$$\frac{d\,U}{d\,T_2} = (-T_1 + T_2) \frac{h_1^3}{3\,E\,I_1} + \frac{T_2\,l_2}{A\,E} + (T_2 - T_3) \frac{h_2^3}{3\,E\,I_2}$$

$$\frac{d\,U}{d\,T_3} = (-T_2 + T_3) \frac{h_2^3}{3\,E\,I_2} + \frac{T_3\,l_3}{A\,E} + (T_3 - P_3) \frac{h_3^3}{3\,E\,I_3}$$

Making the usual substitutions, and equating to D_1, D_2, and D_3, respectively, we have:

$$(B_0 + B_1 + C_1)\,T_1 - B_1\,T_2 = D_1$$

$$-B_1\,T_1 + (B_1 + B_2 + C_2)\,T_2 - B_2\,T_3 = D_2$$

$$-B_2\,T_2 + (B_2 + B_3 + C_3)\,T_3 = D_3$$

Solving for T_1, T_2 and T_3:

$$T_1 = \frac{D_1 + B_1\,T_2}{(B_0 + B_1 + C_1)} \qquad (27)$$

$$T_3 = \frac{D_3 + B_2\,T_2}{B_2 + B_3 + C_3} \qquad (28)$$

$$T_2 = \frac{B_1 (B_2 + B_3 + C_2) D_1 + (B_0 + B_1 + C_1)(B_2 + B_3 + C_3) D_2 + (B_0 + B_1 + C_1) B_2 D_3}{(B_0 + B_1 + C_1)(B_1 + B_2 + C_2)(B_2 + B_3 + C_3) - (B_0 + B_1 + C_1) B_2^2 - B_1^2 (B_2 + B_3 + C_3)} \quad \ldots\ldots(29)$$

If $D_1 = D_2 = D_3 = D$,

$B_1 = B_2 = B_3 = B$,

and $C_1 = C_2 = C_3 = C$,

then

$$T_2' = \frac{(4B + C) D}{(2B + C)^2 - 2B^2} \quad \ldots\ldots\ldots\ldots(28')$$

$$T_1 = T_3 = \frac{D + B T_2}{2B + C} = D \frac{\left(1 + B \dfrac{4B + C}{(2B + C)^2 - 2B^2}\right)}{(2B + C)} \quad \ldots(29')$$

$$= D \left(\frac{(2B + C)^2 - 2B^2 + B(4B + C)}{(2B + C)^3 - 2B^2 (2B + C)}\right)$$

$$= \frac{D (6B^2 + 5BC + C^2)}{(2B + C)((2B + C)^2 - 2B^2))} = \frac{D (3B + C)}{((2B + C)^2 - 2B^2)}$$

$$= \frac{D (3B + C)}{(2B + C)^2 - 2B^2}: \text{ and } T_1 = T_3 = \frac{(3B + C) T_2}{4B + C} \ldots(29'')$$

Application of General Formulas.—The general formulas developed will now be applied to the determination of temperature stresses in the viaduct, the application being based on the following conditions:

First, the three concrete spans monolithic, supported without friction on Piers 3 and 4, and fixed with frictionless pins on Pier 5 and the west abutment. This is Case II.

Second, supported without friction on the abutment and Pier 3, and fixed on Piers 4 and 5. Case III.

Third, supported without friction on Pier 3, and fixed on Piers 4 and 5, and on the abutment. Case IV.

Fourth, supported without friction on the abutment and fixed on Piers 3, 4, and 5. Case V.

Fifth, fixed on all four supports. Case VI.

Proceeding now to the calculation of the various constants involved:

The constants required of each pier are: I, the moment of inertia about an axis perpendicular to the axis of the viaduct; E, the modulus of elasticity of the material; and h the height of the pier from the base to the neutral plane of the spans.

$I = I_A \sin.^2 L + I_B \cos.^2 L$, in which L is the angle made by the axis of the pier with the axis of I (perpendicular to the axis of the via-

duct). I_A is the moment of inertia of the pier about its short axis, and I_B is the moment of inertia about its long axis.

E is taken as 3 000 000 lb. per sq. in., and h is assumed in all cases to be 40 ft.

The constant, $\frac{h^3}{3\,E\,I} = B$, has the following values:

Pier 3, $B_3 = 0.11268 \times 10^{-7}$ in., Log. 2.0518419 — 10;
Pier 4, $B_2 = 0.16333 \times 10^{-7}$ in., Log. 2.2130659 — 10;
Pier 5, $B_1 = 0.24132 \times 10^{-7}$ in., Log. 2.3825920 — 10.

The constants required of each span are:

A = area, in square inches;
l = average length, in inches;
E = its modulus of elasticity;
e = its coefficient of expansion, these being combined into
$c = \frac{l}{A\,E}$ and $D = t\,e\,l$;
t = the range of temperature = 40°;
e = 0.0000055.

Span 4:

$C_3 = 0.1145 \times 10^{-7}$ in., Log. 2.0588055 — 10;
$D_3 = 0.10459$ in., Log. 9.0194902 — 10;
A = 13 960 sq. in. (including steel in terms of concrete).

Span 5:

$C_2 = 0.12798 \times 10^{-7}$ in., Log. 2.1071421 — 10;
$D_2 = 0.11790$ in., Log. 9.0715138 — 10;
A = 13 960 sq. in.

Span 6:

$C_1 = 0.13990 \times 10^{-7}$ in., Log. 2.1458177 — 10;
$D_1 = 0.132$ in., Log. 9.1205739 — 10;
A = 14 300 sq. in.

The foregoing constants are combined into the following terms, for convenience in substitution:

$C_1 + B_1 = 0.38122 \times 10^{-7}$, Log. 2.5811757 — 10;
$C_2 + B_1 + B_2 = 0.53263 \times 10^{-7}$, Log. 2.7264256 — 10;
$C_3 + B_2 + B_3 = 0.39051 \times 10^{-7}$, Log. 2.5916322 — 10;
$B_1^2 = 0.58235 \times 10^{-15}$, Log. 4.7651840 — 20;
$B_2^2 = 0.26677 \times 10^{-15}$, Log. 4.4261318 — 20.

The numerical calculations of the unit stresses in the concrete of Span 5, under the assumption of Case V, are here given to illustrate a convenient method.

Equation (16): $$T_2 = \frac{D_2 (C_3 + B_2 + B_3) + D_3 B_2}{(C_2 + B_1 + B_2)(C_3 + B_2 + B_3) - B_2^2}$$

Log. D_2 $= 9.0715138 - 10$

Log. $(C_3 + B_2 + B_3)$ $= \underline{2.5916322 - 10}$

$11.6631460 - 20 = 0.46041 \times 10^{-8}$.

Log. D_3 $= 9.0194902 - 10$

Log. B_2 $= \underline{2.2130659 - 10}$

$11.2325561 - 20 = \underline{0.17083 \times 10^{-8}}$

Numerator $= 0.63124 \times 10^{-8} = N$.

Log. $(C_2 + B_1 + B_2)$ $= 2.7264256 - 10$

Log. $(C_3 + B_2 + B_3)$ $= \underline{2.5916322 - 10}$

$5.3180578 - 20 = 0.20800 \times 10^{-14}$

Less $B_2^2 = \underline{0.02668 \times 10^{-14}}$

Denominator $= 0.18132 \times 10^{-14} = D$.

Log. N $= 11.8001945 - 20$

Log. D $= \underline{5.2584457 - 20}$

Log. T_2 $= 6.5417488$ $T_2 = 3\,481\,400$ lb.

Log. A $= \underline{4.1448854}$

Log. f_c $= 2.3968634$ $f_c = 250$ lb. per sq. in.

which is the average axial unit stress due to a change in temperature of 40°, and may be either compression or tension. The corresponding stress in the steel is $250 \times 10 = 2\,500$ lb. per sq. in.

The bending stress in Pier 5 for Case VI (that under consideration)

$$= \frac{M y}{I} = \frac{T_3 h_3 y}{I_3} = \frac{3\,481\,400 \times 480 \times 230}{509\,201\,000}$$

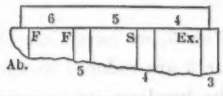
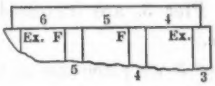
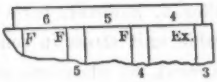
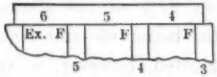
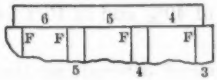
or $f_c = 755$ lb. per sq. in. in the extreme fiber. To this, of course, must be added algebraically the normal stress, in order to get the maximum and minimum.

The bending stresses have been found under the assumption of hinge joints at the piers. The original proposal, however, called for spans integral with the piers. As these spans, under such conditions, would be capable of transmitting considerable moment to the piers, the moment arm, h, of course, would be shortened; but the total stress,

T, at the same time, would be correspondingly increased, owing to the increased stiffness of the pier (see Equation (5)), hence the resulting stresses would not be greatly changed under an assumption of partial or even complete continuity with the supports.

Table 1 shows the arrangements of expansion joints on the viaduct according to Cases II to VI, inclusive, with the corresponding unit axial stresses in the spans, and the bending stresses in the piers.

TABLE 1.

Case.	Diagram.	Average Stress in Spans.			Bending Stress in Piers.		
		6	5	4	5	4	3
II		240	0	0	750	0	0
III		0	160	0	480	390	0
IV		480	400	0	145	1 090	0
V		0	250	300	755	115	600
VI		590	560	430	125	490	860

Ex. = Expansion joint; F = Fixed on pier; S = Sliding on pier.

Now return to the original question: that is, are the stresses due to temperature changes in this viaduct negligible?

Referring to Table 1, it is plain that, under the arrangement of Case VI, with all points of support fixed, a change of 40° Fahr. in

temperature causes severe stresses in all three spans and in Piers 3 and 4. In the spans the stresses are increased from 72 to 92% of the allowable, and the bending stresses caused in the piers are from 21 to 143% of the allowable, assuming the piers to be of 600-lb. concrete. It is interesting to note that the stresses in the spans decrease as those in the piers increase.

TABLE 2.

Span.	Area, in square inches.	Average span, in inches.	Pier.	Assumed height, in inches.	I, in inches.^{4}	Y, in inches.
6	14 300	600	5	480	509 201 000	230
5	13 960	536	4	480	752 342 100	275
4	13 960	480	3	480	1 090 534 000	327

If an expansion joint is placed at Pier 3, we have Case IV. The temperature stresses in Span 4 and Pier 3 are reduced to zero; those in Spans 5 and 6 are reduced; and the bending stresses in Piers 4 and 5, lacking the strong support of Pier 3, are increased, Pier 4 being stressed to 182% of the allowable.

If, instead of placing a joint at Pier 3 we place one at the abutment, the stresses in the spans are still further decreased, as are the bending stresses in Piers 3 and 4. Pier 5, however, takes a heavy increase in stress to 126% of the allowable unit stress in concrete.

If, in addition to a joint at the abutment, we place one at Pier 3, we have Case III, the most favorable arrangement of the five considered, the stress in Span 5 being only 27% of the allowable; but the stresses in the piers are still high, being 65 and 80% of the maximum allowable. They can be neglected, however, as the direct stress is small.

Case II produces a stress in Span 6 of 40% and in Pier 5 of 125% of the allowable unit stress.

The problem is now resolved into one of determining the relative economy of expansion joints and of reducing the maximum live-load and dead-load unit stresses by increasing the section. Case III is the only arrangement with a restrained span which would not produce prohibitive stresses in at least one of the piers. It is also to be pointed out that increasing the section of Span 5 will increase the stresses in Piers 4 and 5 in direct proportion.

The plan finally adopted was to put expansion joints at the abutment and Pier 3 and a sliding joint over Pier 5. The combination spans of the east end of the viaduct are all provided with expansion joints.

Mr. I. L. Simmons and C. E. Smith, M. Am. Soc. C. E., the Bridge Engineers, respectively, of the Chicago, Rock Island and Pacific, and the St. Louis, Iron Mountain and Southern Railroads, were responsible for the design of this bridge.

DISCUSSION

Mr. Martin. CHARLES W. MARTIN,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer has read this paper with some interest, but, save for a "look in" at the general principles, has made no attempt to follow the theoretical analysis.

In the design of reinforced concrete bridges and viaducts for the City of St. Louis, the writer, from time to time, has given considerable thought to the matter of expansion joints and stresses generated by changes of temperature. At the first glance through Mr. Gregg's paper, he was somewhat amazed at the high stress units tabulated for the structure, with the several arrangements of joints and bearings as described.

In a series of spans, temperature changes generate two distinct combinations of forces: bending combined with shear in the piers, and axial forces combined with bending in the deck. The magnitude of these forces varies directly with the moment of inertia of the pier sections in any given system of spans and piers. Thus it happens that, as in the case of the design of flat arch ribs subject to temperature changes, one frequently finds that, to reduce the stresses in the structure, he must make certain parts more flexible. In the design of a series of spans and piers, relief from high temperature stresses is obtained by a reduction of the pier sectioning.

The structure which Mr. Gregg has selected to illustrate his theory has an extremely inflexible arrangement of skewed piers, and to this is due, in accordance with his analysis, the unusually high temperature stresses. Here, the characterization "unusually high" is used advisedly, particularly as regards the axial stresses for the spans in his Cases 4 and 6 of Table 1, for if one considered any span absolutely restrained from change in length, the temperature stresses would not greatly exceed those reported.

Although the principle of Mr. Gregg's theory may be quite useful, the writer thinks he was unfortunate in the selection of a trial structure. With a structure in which the piers have such a skew, the critical guess one has to make is as to the direction of the principal distortions. Mr. Gregg has assumed that these act parallel to the longitudinal axis of the structure, whereas actually to do the "least work", these principal distortions will act in some oblique direction which will depend on the skew of the piers and the lateral restraint at the ends of the outmost spans.

Mr. Gregg. TRESHAM D. GREGG,† ASSOC. M. AM. SOC. C. E. (by letter).—The writer quite agrees with Mr. Martin that the stresses found are unusually high. In defense, however, of his selection of an example to

* St. Louis, Mo.

† Minneapolis, Minn.

illustrate the principles involved, he wishes to say that it is his experience that skewed piers are frequently necessary, and that massive piers are sometimes more economical than light, flexible bents, and always considerably safer where there is danger from collision with derailed cars or locomotives. It is freely admitted that the theory developed has small practical application, except in the type of bridge selected. Mr. Gregg.

The writer also agrees with Mr. Martin's "guess" that the distortions involving least work will be in a direction oblique to the axis of the bridge. The calculation of the true stresses in such a combination would be extremely complicated. In order to find the true state of stress in all the members—in the case where all supports are fixed—consideration must be given to the work of expansion of the several spans against piers which have a variable moment of inertia, depending on the extent to which they yield in a lateral direction; to the bending of the spans, particularly the end spans, due to the lateral movement of the piers; and to the work of torsion of the piers and bending of the spans, due to the change in angle of skew, brought about also by the lateral deflection of the piers.

The writer has reason to think that this lateral movement of the piers is a very minute quantity. If it should amount to one-half of what it would be if the spans were pivoted on the piers and the abutment ends were free to move, bending stresses in the span would be set up, which would be, if anything, greater than the axial stresses calculated under the original assumption, and bending stresses in the piers—perhaps one-fourth in amount of those tabulated in this paper—would nevertheless be combined with high torsion stresses, leaving the situation, in the writer's opinion, about as bad as before.

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Paper No. 1373

THE FLOW OF WATER IN IRRIGATION CHANNELS*

By George Henry Ellis, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. H. B. Muckleston, Harold F. Gray, S. T. Harding, Allen Hazen, R. B. Sleight, W. G. Hoyt, William S. Aldrich, M. J. Orbeck and C. N. Ward and W. O'B. Henderson, and George Henry Ellis.

Synopsis.

This paper presents a study of some data recently published as *Bulletin No. 194* of the United States Department of Agriculture. An exponential formula is deduced for the flow of water, $V = C\,R^{0.69}\,S^{0.5}$, in which the coefficient, C, varies from about 40 to 140, depending on the roughness of the channel. For general conditions, the following formulas are submitted:

For concrete channels..... $V = 105\,R^{0.69}\,S^{0.5}$.
For wooden channels..... $V = 100\,R^{0.69}\,S^{0.5}$.
For earth canals..... $V = 60\,R^{0.69}\,S^{0.5}$.

Incidentally, this paper shows the need of care in the selection of a value for the coefficient of roughness. For example, Experiment No. 4, Table 1, was conducted under conditions which gave a value of n, in Kutter's formula, equal to 0.0108. Entering Fig. 3 with this value of n, it is observed that from 140 to 145 would have been a better value for C (in the writer's formula) than the 123.7 which was used. With $C = 140$ we get a velocity of 20.1.

* Presented at the meeting of March 15th, 1916.

The data on which this paper is based were taken on ditches with hydraulic radii up to about 4.0 and slopes ranging from about 0.01 to 0.0001.

Table 3 gives the 0.69th powers of radii.

Considerable interest has been manifested in this subject during recent years, and experimental work has been done by the U. S. Reclamation Service, the various State Experiment Stations, the U. S. Office of Experiment Stations, and by individuals. The Office of Experiment Stations has collected and published as a bulletin all recent available data on this subject.* This paper uses the tabulated data on pages 19 to 27 of that bulletin for the purpose of finding a simpler formula for the flow of water.

Different investigators use different methods for determining the hydraulic elements of a ditch. Those used by the Office of Experiment Stations are described in detail in the bulletin mentioned. Briefly, a stretch of canal was chosen, about 1 000 ft. long, and with conditions as nearly uniform as possible throughout its length. Bench-marks were set at each end, and several cross-sections were taken in order to determine the areas and perimeters. The discharge was measured generally with a current meter at the center of the reach chosen, and referred to readings of the water surface on the bench-marks at each end.

The claim has been made† that water can be measured to within 1% with a current meter. The writer believes that results within 2 or 3% can be obtained with this instrument under favorable conditions, but doubts whether dependence can be placed on it within less than about 5% for all sorts of conditions. Hoyt and Grover give tables‡ which seem to indicate that even this is optimistic. In setting the bench-marks for the determination of the slope, there is another chance for error. Assuming an allowable error (in feet) of $0.017\sqrt{\text{distance, in miles}}$, gives an error of 0.0074 in 1 000 ft. A slope of 0.0001—which is about as flat as any canal is built—has a fall of 0.1 ft. in that distance. The probable error in levels would then be 7.4% on a ditch having a slope of 0.0001.

* *Bulletin No. 194*, U. S. Department of Agriculture, May 10th, 1915.

† In a paper on Irrigation Hydrography, read before a conference of irrigation officials at North Yakima, Wash., December 7th and 8th, 1910.

‡ "River Discharge," pp. 111 and 112.

Water runs in a canal by merely sliding down hill—a process so simple that to make experiments and write papers on it seems almost absurd—and yet, because of the many things affecting friction, not very much is known about the flow of water, and this little paper will be worth while if it does no more than call attention to recently published data and start a discussion which may lead to the adoption of a more simple formula than that now in use.

In 1775 Chezy proposed his well-known formula for the flow of water, $V = C\sqrt{R\,S}$.

In 1869 Ganguillet and Kutter enlarged this by their empirical value of C:

$$V = \left\{ \frac{\frac{1.811}{n} + 41.6 + \frac{0.00281}{s}}{1 + \left[41.6 + \frac{0.00281}{s}\right]\frac{n}{\sqrt{R}}} \right\} \sqrt{R\,S}.$$

In 1897 Bazin attempted to shorten this by proposing a simpler value of C:

$$V = \frac{157.6}{1 + \frac{\gamma}{\sqrt{R}}} \sqrt{R\,S}.*$$

Engineers, however, had become so accustomed to thinking in terms of n, that they were loth to change. In 1911 Johnston and Goodrich proposed using an exponential formula of the form, $V = CR^pS^q$, and gave values of C and p, making q uniformly equal to 0.5 for simplicity.† This type of formula has also been advanced by Manning for open channels:

$$V = \frac{1.49}{n} R^{0.67} S^{0.5},‡$$

and by others for closed conduits.

For example,§ Lampe..... $V = 77.68\ D^{0.694}\ S^{0.555}$;

Flamant..... $V = (76.28 \text{ to } 86.38)\ D^{\frac{5}{7}}\ S^{\frac{4}{7}}$;

Williams.... $V = 67.7\ D^{0.668}\ S^{0.535}$;

Moritz..... $V = 77\ D^{0.7}\ S^{0.555}$.

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* Parker, "The Control of Water", p. 474.

† *Engineering Record*, Vol. 64, p. 542, November 4th, 1911.

‡ Parker, "The Control of Water", p. 472.

§ *Engineering Record*, Vol. 68, p. 669, December 13th, 1913.

A formula of this type is selected for the purposes of this paper, as it is easily deduced by plotting on logarithmic paper, as follows:

$$V = C\,R^p\,S^{0.5}.$$

$$\frac{V}{S^{0.5}} = C\,R^p.$$

$$\log.\ \frac{V}{S^{0.5}} = \log.\ C + p\ \log.\ R,$$

which is the equation of a straight line.

In recommending values of n for channels, the author of *Bulletin No. 194*, Frederick C. Scobey, Assoc. M. Am. Soc. C. E., has classified them according to different types. His classification has been adhered to in copying the data into Table 1, but only five of the twenty-three columns of data, which are of immediate use in this paper, are given. The first, second, fourth, fifth and sixth columns of Table 1 were copied from the bulletin, and the third, seventh, and eighth were computed by the writer. These data were plotted on Plates LIX and LX and Figs. 1 and 2, each point being marked for identification with its assigned value of n. Curves (or rather straight lines) were drawn through these points for each type of channel, and from these the values of the coefficient, C, and the angle, α, in Table 2 were taken.

TABLE 1.

Concrete, Type I.

"$n = 0.012$ for the highest grade of material and workmanship and exceptionally good conditions. The surface of the lining to be as smooth to the hand as a troweled sidewalk. The expansion joints to be so well covered that they practically fulfill the same condition. The climate and water to be such that moss does not accumulate to any great extent. The water to be practically free from shifting material. The alignment to be composed of long tangents joined by spiraled curves, while the interior of the channel must be of uniform dimensions, true to grade throughout the cross-section."

No. of experiment.	Measured velocity.	Velocity by formula: $V = 123.7\,R^{0.69}\,S^{0.5}$.	Radius.	Slope.	n.	$S^{0.5}$.	$C\,R^p$.
1	5.48	4.92	5.23	0.0001611	0.0101	0.0127	431.0
4	20.62	17.77	1.30	0.0144	0.0108	0.1200	171.8
6	2.81	2.46	0.83	0.00051	0.0108	0.0226	124.3
7	2.71	2.44	0.82	0.00051	0.0111	0.0226	119.9
8	4.74	4.39	1.37	0.00082	0.0113	0.0286	165.5
9	15.78	14.29	0.94	0.01459	0.0115	0.1207	130.7
11	4.15	3.89	2.81	0.0002371	0.0110	0.0154	269.5
12	3.65	3.71	2.54	0.0002508	0.0121	0.0158	231.0
14	4.14	4.34	2.91	0.000283	0.0124	0.0168	246.4
16	15.05	15.09	0.29	0.08244	0.0124	0.2871	52.6
18	19.70	19.20	0.86	0.02971	0.0123	0.1723	114.3
19	12.87	13.03	0.49	0.02978	0.0125	0.1725	74.6

TABLE 1.—(*Continued.*)

CONCRETE, TYPE II.

"$n = 0.013$ for construction as in Type I, but with curves as in the usual mountain canyon. Same construction and alignment as in type I, but with a small amount of sand or débris in water. Construction as in type III, but in very favorable alignment or for water that carries a small amount of fine silt that will eventually form a slick coat."

No. of experiment.	Measured velocity.	Velocity by formula: $V = 114.0\ R^{0.69}\ S^{0.5}$.	Radius.	Slope.	n.	$S^{0.5}$.	$C\ RP$.
2	3.81	3.85	2.45	0.0003333	0.0130	0.0182	209.0
10	5.01	4.88	1.31	0.00126	0.0128	0.0355	141.0
13	3.65	3.64	2.54	0.0002837	0.0129	0.0168	217.0
15	2.45	2.47	1.30	0.000329	0.0132	0.0181	135.0
17	6.18	6.68	0.10	0.08244	0.0130	0.2871	21.5
20	2.06	2.06	0.58	0.0007	0.013	0.0264	78.0
21	7.10	7.19	2.13	0.0014	0.0132	0.0374	189.8

CONCRETE, TYPE III.

"$n = 0.014$ for linings made by good construction under favorable conditions. The surface to be as left by smooth-jointed forms or to be roughly troweled. Joints to be good, but causing some retardation. Alignment about equal in curves and tangents, with no spirals between. The bed to be clean and sides free from rough deposits."

The authors of the bulletin believe this is the proper value to use for most linings on moderate-sized channels.

No. of experiment.	Measured velocity.	Velocity by formula: $V = 105.8\ R^{0.66}\ S^{0.5}$.	Radius.	Slope.	n.	$S^{0.5}$.	$C\ RP$.
5	19.13	18.78	1.36	0.0206	0.0140	0.1435	133.2
22	7.15	7.46	2.12	0.00177	0.0142	0.0420	170.0
24	16.38	.....	0.22	0.02056	0.0139	0.1434	114.2†
25	8.21	8.57	5.43	0.0006389	0.0138	0.0252	325.0
26	3.82	3.86	2.34	0.000413	0.014	0.0203	186.1
27	3.89	3.50	1.50	0.000626	0.0144	0.0250	135.6†
30	2.55	2.62	1.25	0.0004497	0.0143	0.0212	120.3

CONCRETE, TYPE IV.

"$n = 0.015$ for construction as in type III, but with sharp curves and clean bottom or moderate curves and much débris on the bottom but clean-cut sides."

No. of experiment.	Measured velocity.	Velocity by formula: $V = 98.4\ R^{0.69}\ S^{0.5}$.	Radius.	Slope.	n.	$S^{0.5}$.	$C\ RP$.
3	4.68	4.93	3.88	0.0003875	0.0154	0.0197	237.5
28	3.34	3 24	1.50	0.000619	0.0146	0.0249	134.1
29	3.94	4.08	2.07	0.000629	0.0154	0.0251	157.0
31	3.86	3.87	1.94	0.00062	0.0149	0.0249	155.0

TABLE 1.—(*Continued.*)

CONCRETE, TYPE V.

"$n = 0.016$ for concrete as constructed by the average gang of laborers, using forms that leave prominent lines at the cracks, no finish coat being applied. Bed to have the usual small amount of rock fragments and patches of sand and gravel. Average amount of curvature. In climates where a rough deposit accumulates, as in southern California, a lining that originally had a value of n about 0.013 quickly assumes about this type."

No. of experiment.	Measured velocity.	Velocity by formula: $V = 92.1\ R^{0.69}\ S^{0.5}$.	Radius.	Slope.	n.	$S^{0.5}$.	$C\ RP$.
32	4.71	4.52	1.41	0.00151	0.0155	0.0388	121.4
33	15.52	15.47	0.51	0.07180	0.0158	0.2679	57.9
35	3.74	3.98	0.84	0.002375	0.0155	0.0488	76.6
36	2.62	2.62	0.82	0.00106	0.0157	0.0326	80.4
37	1.86	1.89	0.54	0.00069	0.0160	0.0315	59.0

CONCRETE, TYPE VI.

"$n = 0.017$ for roughly coated linings with uneven joints. This value also is applicable where rough deposits accumulate on the sides and conditions of alignment are poor."

No. of experiment.	Measured velocity.	Velocity by formula: $V = 86.0\ R^{0.69}\ S^{0.5}$.	Radius.	Slope.	n.	$S^{0.5}$.	$C\ RP$.
34	11.32	11.83	0.38	0.07230	0.0171	0.2689	42.1
39	2.27	2.24	0.98	0.0007	0.0167	0.0264	86.0
40	1.38	1.44	0.52	0.000694	0.0171	0.0263	52.5
41	3.58	3.65	1.38	0.001157	0.0174	0.0340	105.3

CONCRETE, TYPE VII.

"$n = 0.018$ for very rough concrete with sharp curves and deposits of gravel and moss. A broken gradient, irregular cross-section, and the like, contribute to such a high value of n."

No. of experiment.	Measured velocity.	Velocity by formula: $V = 80.3\ R^{0.69}\ S^{0.5}$.	Radius.	Slope.	n.	$S^{0.5}$.	$C\ RP$.
23	6.94	7.11	2.15	0.00273	0.0189	0.0522	133.0
38	1.71	1.89	0.58	0.001184	0.0192	0.0344	49.7
42	1.67	1.65	1.22	0.0003208	0.0176	0.0179	93.3
43	2.89	3.01	0.98	0.001444	0.0188	0.0380	76.0
44	2.83	2.94	0.95	0.001449	0.0188	0.0380	74.5
45	2.94	2.90	1.94	0.000525	0.0177	0.0229	128.4
46	3.02	3.16	1.90	0.000639	0.0187	0.0253	119.4
47	2.85	2.82	1.63	0.000629	0.0176	0.0251	113.5
48	2.87	3.02	1.62	0.000729	0.0190	0.0270	106.3
49	2.10	2.26	0.88	0.000950	0.0192	0.0308	68.2
50	2.04	2.40	0.91	0.001021	0.0206	0.0320	63.8
51	2.27	2.70	1.63	0.000574	0.0211	0.0240	94.6
52	1.22	1.57	0.70	0.00063	0.0218	0.0251	48.6
53	1.07	1.39	0.62	0.0005839	0.0220	0.0242	44.2
54	1.88	2.38	1.02	0.000851	0.0221	0.0292	64.4
55	1.79	2.44	1.60	0.000482	0.0231	0.0220	81.4
56	1.96	3.20	1.49	0.00092	0.0284	0.0303	64.6

TABLE 1.—(*Continued.*)

WOODEN CHANNELS, TYPE I.

"$n = 0.012$ for well-constructed, clean flumes with surfaced lumber for both siding and battens. All lumber to run longitudinally. Alignment to consist of long tangents with gentle curves between. Construction to be such that the grade line will remain uniform, preventing sags and wavy alignment. A flume without battens may have a slightly lower value of n, but this difference will be inappreciable if the added length of the wetted perimeter due to the battens is considered in the design. Some very smooth grades of roofing materials used as linings also give a slightly lower value of n, but not enough to consider. If flumes are calked with oakum or other stuffing, care must be used that none projects into the water section if a high degree of efficiency is to be maintained."

No. of experiment.	Measured velocity.	Velocity by formula: $V = 123.7\ R^{0.69}\ S^{0.5}$.	Radius.	Slope.	n.	$S^{0.5}$.	$C\ R^{p}$.
57	4.49	3.85	2.28	0.0003108	0.0103	0.0176	255.0
58	3.23	3.03	1.94	0.00024	0.0112	0.0155	208.4
59	5.79	5.46	1.35	0.001288	0.0115	0.0359	161.3
60	5.08	4.68	1.66	0.000713	0.0112	0.0267	190.3
61	3.80	3.53	1.11	0.000710	0.0114	0.0266	142.8
62	5.96	6.12	1.96	0.000965	0.0123	0.0311	191.6
64	9.12	8.82	1.90	0.00210	0.0117	0.0458	199.0
65	7.80	7.43	1.47	0.00213	0.0118	0.0461	169.0
69	14.31	...	0.23	0.01054	0.0122	0.1026	139.4?
70	2.32	2.44	1.92	0.00016	0.0125	0.0126	184.0

WOODEN CHANNELS, TYPE II.

"$n = 0.013$ for well-constructed, clean flumes of surfaced lumber and battens, following mountain contours, where the alignment will consist of about equal gentle curves and tangents. This value will also apply to flumes with alignment and grade as described in type I, but with vertical battens at intervals, with projecting calking or a slight amount of hardened asphalt or other water-proofing retarding the velocity."

No. of experiment.	Measured velocity.	Velocity by formula: $V = 114.0\ R^{0.69}\ S^{0.5}$.	Radius.	Slope.	n.	$S^{0.5}$.	$C\ R^{p}$.
63	5.81	5.61	2.12	0.000858	0.0126	0.0293	198.3
66	7.30	6.94	1.49	0.00213	0.0126	0.0462	158.0
67	7.93	7.66	1.72	0.00213	0.0127	0.0462	171.6
71	2.60	2.57	1.80	0.000225	0.0127	0.0150	173.3
72	2.08	2.09	1.96	0.000133	0.0129	0.0115	180.0
77	3.71	3.66	0.99	0.001044	0.0131	0.0323	114.8

WOODEN CHANNELS, TYPE III.

"$n = 0.014$ for flumes of very smooth interior, but with many bends or sharp curves. This value also applies to those of type I with a location such that a slight amount of hillside débris is unavoidable.

TABLE 1.—(*Continued.*)

Construction of type I, but with cracks poured with any water-proofing material that hardens in tapering drops, is also liable to come in this type. For type I except that unsurfaced lumber is used."

No. of experiment.	Measured velocity.	Velocity by formula: $V = 105.8\ R^{0.69}\ S^{0.5}$.	Radius.	Slope.	n.	$S^{0.5}$.	$C\ R^{p}$.
73	2.67	2.60	2.24	0.00020	0.0135	0.0141	189.0
74	1.64	1.66	0.40	0.000868	0.0138	0.0295	55.6
75	6.31	6.17	1.66	0.00169	0.0141	0.0411	151.0
76	5.46	5.44	0.87	0.0032	0.0142	0.0566	96.5
78	3.44	3.57	1.06	0.001048	0.0145	0.0324	106.2

WOODEN CHANNELS, TYPE IV.

"n = 0.015 for flumes of unplaned lumber, but otherwise as of type II."

The authors of the bulletin believe this is about the value to use for the usual grade of construction in a mountain canyon where the flume will get about the usual grade of maintenance, with repairs made with irregular-shaped scraps of boards.

No. of experiment.	Measured velocity.	Velocity by formula: $V = 98.4\ R^{0.69}\ S^{0.5}$.	Radius.	Slope.	n.	$S^{0.5}$.	$C\ R^{p}$.
68	5.45	5.24	1.23	0.00213	0.0149	0.0462	118.0
79	3.34	3.34	1.08	0.001085	0.0150	0.0322	103.7
80	3.34	3.37	1.08	0.001054	0.0152	0.0325	102.8
81	1.54	1.55	0.93	0.000274	0.0150	0.0166	93.0
82	3.13	3.18	1.64	0.00053	0.0153	0.0230	136.0
84	4.08	4.03	0.90	0.001934	0.0149	0.0440	92.7

WOODEN CHANNELS, TYPE V.

"n = 0.016 for flumes of type IV where sharp bends rather than curves are installed. For flumes lined with rough roofing material and for the ordinary grade of construction on a flume that is built and generally left to care for itself. The kind of organization that is to operate the flume will determine this factor."

No. of experiment.	Measured velocity.	Velocity by formula: $V = 92.1\ R^{0.69}\ S^{0.5}$.	Radius.	Slope.	n.	$S^{0.5}$.	$C\ R^{p}$.
83	3.06	2.95	1.62	0.00053	0.0155	0.0230	133.0
85	3.99	3.93	0.93	0.002019	0.0156	0.0449	88.8
86	3.61	3.55	1.00	0.001479	0.0157	0.0385	93.7
87	4.35	3.87	0.87	0.002145	0.0157	0.0463	93.9
88	1.30	1.31	0.66	0.0003615	0.0159	0.0190	68.4
89	1.08	1.10	1.94	0.0000568	0.0163	0.0075	143.5
90	6.11	6.61	0.59	0.01069	0.0167	0.1034	59.1
91	1.60	1.70	1.04	0.0003249	0.0167	0.0180	88.8
92	0.65	0.83	0.33	0.00038	0.0184	0.0195	33.3
93	1.22	1.50	2.30	0.000684	0.0191	0.0092	133.0
94	3.58	4.57	0.72	0.003915	0.0196	0.0626	57.2
95	2.97	3.80	2.86	0.0004	0.0201	0.0200	148.5
96	1.56	2.08	0.86	0.0006299	0.0202	0.0251	62.2
97	2.46	3.41	2.94	0.00081	0.0217	0.0176	140.0

TABLE 1.—(*Continued.*)

METAL FLUMES, TYPE I.

$n = 0.011$ for flumes "having countersunk joints between the various sheets of metal so that there is practically no added roughness presented to the water"; "in favorable alignment and clean."

No. of experiment.	Measured velocity.	Velocity by formula: $V = 137.0\ R^{0.69}\ S^{0.5}$.	Radius.	Slope.	n.	$S^{0.5}$.	$C\ R^{p}$.
98	6.01	4.97	0.37	0.0052	0.0099	0.0721	84.2
99	5.34	4.68	0.61	0.0023	0.0101	0.0480	111.4
100	1.66	1.56	0.38	0.0005	0.0106	0.0223	74.4
101	2.37	2.48	0.41	0.00113	0.0117	0.0336	70.5
102	2.55	2.53	0.41	0.00117	0.0112	0.0342	74.5
103	2.88	2.60	0.32	0.00175	0.0122	0.0418	68.9
104	4.40	4.97	0.69	0.0022	0.0126	0.0469	93.8

METAL FLUMES, TYPE II.

$n = 0.015$ for flumes "having joints that project into the water section presenting a shoulder every few feet that effectually retards the velocity"; "in favorable alignment and clean."

No. of experiment.	Velocity.		Radius.	Slope.	n.	$S^{0.5}$.	$C\ R^{p}$.
105	1.82	.....	0.32	0.00120	0.0127	0.0346	52.5
106	1.43	.....	0.84	0.00020	0.0129	0.0141	101.0
107	1.38	.....	0.91	0.000175	0.013	0.0132	104.5
108	1.68	.....	0.31	0.00130	0.0139	0.0360	46.6
109	5.35	.....	1.07	0.00386	0.0179	0.0621	86.1
110	5.77	.....	0.92	0.00587	0.0177	0.0733	78.7
111	5.10	.....	0.84	0.00411	0.0166	0.0641	79.6

METAL FLUMES, TYPE III.

$n = 0.022$ for flumes "having regular corrugations at right angles to the axis of the flume"; "in favorable alignment and clean".

No. of experiment.	Velocity.		Radius.	Slope.	n.	$S^{0.5}$.	$C\ R^{p}$.
112	1.92	.....	1.04	0.000892	0.0222	0.0299	64.2

EARTH CANALS, TYPE I.

"$n = 0.016$ for excellent conditions of earth channels. The velocity to be so low that a slick deposit of silt may accumulate, or the natural material be such as to become smooth when wet. The influence of vegetation at the edges to be a minimum. The water to be free from moss and other aquatic growth. The alignment to be free from bends and sharp curves."

TABLE 1.—(*Continued.*)

No. of experiment.	Measured velocity.	Velocity by formula: $V = 92.1\ R^{0.69}\ S^{0.5}$.	Radius.	Slope.	n.	$S^{0.5}$.	$C\ RP$.
118	4.75	3.05	3.88	0.00017	0.012	0.0130	365.0
119	2.56	2.09	2.40	0.000154	0.0130	0.0124	206.0
120	2.22	2.32	2.62	0.00017	0.0164	0.0130	170.0
121	3.62	3.04	2.49	0.00031	0.0134	0.0176	205.5
122	1.04	0.90	0.86	0.00012	0.0135	0.0109	95.0
123	2.36	2.32	1.86	0.00027	0.0155	0.0164	143.5
124	2.02	2.10	1.60	0.000273	0.0164	0.0165	122.0
125	1.86	1.95	1.13	0.00038	0.0165	0.0195	95.4
126	1.28	1.43	2.20	0.0000804	0.0166	0.0090	142.5
127	0.93	1.06	0.99	0.0001341	0.0170	0.0116	80.2
128	2.94	2.99	2.84	0.00025	0.0170	0.0158	186.0
130	2.45	2.70	2.60	0.000230	0.0174	0.0152	161.5
131	1.96	2.18	1.11	0.00049	0.0176	0.0221	88.5
132	1.14	1.35	0.47	0.000617	0.0180	0.0248	46.0
134	2.30	2.66	2.12	0.000295	0.0181	0.0172	133.7
135	2.72	3.14	2.04	0.000438	0.0182	0.0209	130.0
136	2.58	3.00	1.20	0.00083	0.0184	0.0288	89.6

Earth Canals, Type II.

"$n = 0.020$ for well-constructed canals in firm earth or fine, packed gravel where velocities are such that silt may fill the interstices in the gravel. The banks to be clean-cut and free from disturbing vegetation. The alignment to be reasonably straight."

No. of experiment.	Measured velocity.	Velocity by formula: $V = 70.0\ R^{0.69}\ S^{0.5}$	Radius.	Slope.	n.	$S^{0.5}$.	$C\ RP$.
129	2.46	2.29	1.73	0.00050	0.0194	0.0224	109.8
133	0.97	0.93	0.50	0.000460	0.0186	0.0214	45.3
137	1.46	1.32	2.34	0.000111	0.0186	0.0105	138.5
138	1.88	1.75	2.79	0.000152	0.0193	0.0123	152.5
139	2.10	1.94	2.8	0.0001867	0.0194	0.0136	154.0
140	1.67	1.63	2.85	0.000127	0.0199	0.0113	148.0
141	2.56	2.34	2.41	0.00033	0.0188	0.0182	140.6
142	0.42	0.45	0.71	0.000067	0.0192	0.0082	51.2
143	1.14	1.13	0.50	0.00068	0.0194	0.0261	43.7
144	1.94	1.82	1.07	0.00062	0.0195	0.0249	77.9
145	1.85	1.77	2.20	0.000215	0.0196	0.0147	126.0
146	1.08	1.08	1.40	0.00015	0.0197	0.0122	88.5
147	1.51	1.49	1.69	0.00022	0.0200	0.0148	102.0
148	1.01	1.05	0.52	0.00056	0.0201	0.0237	42.6
149	0.67	0.71	1.04	0.000098	0.0202	0.0099	67.7
150	0.54	0.65	0.14	0.00135	0.0204	0.0367	14.7
151	2.56	2.52	2.85	0.000308	0.0204	0.0175	146.0
152	0.91	0.87	0.57	0.00034	0.0204	0.0184	49.5
153	1.74	1.87	0.33	0.00334	0.0205	0.0578	30.1
154	1.17	1.21	0.95	0.00032	0.0206	0.0179	65.3
155	2.00	2.02	2.03	0.00312	0.0208	0.0177	113.0
156	2.14	2.19	1.74	0.00046	0.0211	0.0214	100.0
157	1.59	1.63	2.03	0.00020	0.0211	0.0141	112.5
158	0.78	0.84	0.88	0.000175	0.0212	0.0132	59.0

TABLE 1.—(*Continued.*)

EARTH CANALS, TYPE III.

"n = 0.0225, although carried to one more significant figure, is given for the reason that it has long been used for this type and the tests do not disclose any reason for changing. This value for the average well-constructed canal in material which will eventually have a medium-smooth bottom, with graded gravel, grass on the edges and average alignment or silt at both sides of the bed and scattered stones in the middle, or a smooth bottom with an average amount of grass and roots forming the sides. Hardpan in good condition, clay and lava-ash soil take about this value."

No. of experiment.	Measured velocity.	Velocity by formula: $V = 60.2\ R^{0.69}\ S^{0.5}$.	Radius.	Slope.	n.	$S^{0.5}$.	$C\ RP$.
159	2.99	2.22	1.52	0.00077	0.0213	0.0277	89.8
160	0.43	0.41	2.90	0.0000107	0.0216	0.003275	131.2
161	2.09	1.90	1.80	0.00044	0.0216	0.0210	99.5
162	1.84	1.26	1.11	0.0003795	0.0217	0.0195	68.7
163	1.48	1.37	1.60	0.00027	0.0218	0.0164	90.0
164	0.82	0.76	2.99	0.0000345	0.0219	0.0059	139.0
165	1.55	1.44	1.88	0.00024	0.0219	0.0155	100.0
167	1.24	1.18	1.06	0.000357	0.0220	0.0189	65.6
168	1.66	1.56	1.52	0.00038	0.0220	0.0195	85.1
169	1.47	1.37	1.66	0.00026	0.0220	0.0161	91.2
170	3.86	3.60	1.42	0.00220	0.0221	0.0469	82.3
172	1.44	1.37	1.06	0.000481	0.0221	0.0219	65.7
173	2.08	1.93	3.69	0.0001682	0.0221	0.0130	160.0
174	2.00	1.86	2.13	0.000335	0.0221	0.0183	109.3
175	3.12	2.89	2.16	0.000798	0.0222	0.0282	110.4
176	1.19	1.13	1.52	0.00020	0.0224	0.0141	84.0
177	1.15	1.13	0.91	0.00040	0.0224	0.0200	57.5
178	2.09	1.97	2.13	0.000377	0.0225	0.0194	107.5
179	1.63	1.56	1.97	0.000262	0.0226	0.0162	100.6
180	1.78	1.75	1.11	0.0007365	0.0228	0.0271	65.5
181	0.71	0.75	0.48	0.00043	0.0229	0.0207	34.3
182	1.19	1.22	0.65	0.00075	0.0230	0.0274	43.4
183	1.38	1.40	0.71	0.000875	0.0230	0.0296	46.6
184	1.90	1.86	1.93	0.0003859	0.0231	0.0196	96.7
185	1.01	1.00	1.83	0.00012	0.0232	0.0109	92.1
186	2.82	2.82	1.65	0.001105	0.0236	0.0332	84.8

EARTH CANALS, TYPE IV.

"n = 0.025 for canals where the retarding influence of moss, growths of dense grass near the edges, or scattered cobbles begins to show. The value of n in earth channels where the maintenance is neglected commences at this value and rapidly goes up. This is a good value to use in the design of small head ditches or a small ditch to serve but one or two farms."

TABLE 1.—(*Continued.*)

No. of experiment.	Measured velocity.	Velocity by formula: $V = 52.8\ R^{0.69}\ S^{0.5}$.	Radius.	Slope.	n.	$S^{0.5}$.	$C\ R^{p}$.
187	1.90	1.74	0.71	0.00175	0.0238	0.0418	45.5
188	2.93	2.74	0.55	0.00616	0.0238	0.0785	37.3
189	1.97	1.80	1.62	0.0006	0.0246	0.0245	80.4
190	0.34	0.36	0.58	0.0001014	0.0246	0.0100	33.7
191	1.61	1.57	0.65	0.0016	0.0247	0.0400	40.2
192	1.00	1.04	0.43	0.001246	0.0248	0.0353	28.3
193	1.82	1.72	2.89	0.000248	0.0258	0.0157	115.5
194	1.75	1.71	1.29	0.000737	0.0259	0.0271	64.5
195	2.35	2.21	3.02	0.00038	0.0259	0.0195	120.5
196	0.84	0.92	0.35	0.0013	0.0260	0.0360	23.3
197	1.61	1.55	2.04	0.00032	0.0260	0.0179	90.0
198	1.72	1.72	1.20	0.000831	0.0262	0.0288	59.7
199	0.99	1.01	1.27	0.000263	0.0262	0.0162	61.1
200	2.67	2.57	2.38	0.00072	0.0264	0.0268	99.6
201	1.68	1.70	1.52	0.00058	0.0266	0.0241	69.7
202	1.57	1.58	1.57	0.0004783	0.0267	0.0219	71.7
203	1.69	1.70	1.69	0.00050	0.0267	0.0224	75.4
204	1.14	1.20	1.03	0.000493	0.0269	0.0222	51.3
205	1.41	1.54	0.59	0.001758	0.0270	0.0149	33.6
206	1.45	1.52	1.02	0.000812	0.0271	0.0285	50.9

Earth Canals, Type V.

"$n = 0.030$ for canals subject to heavy growths of moss or other aquatic plants. Banks irregular or overhanging with dense rootlets. Bottom covered with large fragments of rock, or bed badly pitted by erosion. Values of n between 0.025 and 0.030 also cover the condition where the velocity is so high that cobbles are kept clean and unpacked in the center of the canal, but silt deposits near the sides. For values above 0.030 the channel is much choked with vegetation, very irregular, crooked, overhung with dragging trees and grasses, or there is some other condition that should not be allowed to exist in a well-kept system."

No. of experiment.	Measured velocity.	Velocity by formula: $V = 44.4\ R^{0.69}\ S^{0.5}$.	Radius.	Slope.	n.	$S^{0.5}$.	$C\ R^{p}$.
166	1.56	1.38	2.04	0.00036	0.0281	0.0190	82.1
171	1.39	1.88	0.60	0.00362	0.0390	0.0602	23.1
207	1.23	1.13	0.83	0.000842	0.0277	0.0290	42.4
208	1.04	1.04	0.75	0.00082	0.0294	0.0286	36.3
209	1.32	1.59	0.47	0.003679	0.0274	0.0606	21.8
210	1.20	1.14	0.66	0.001168	0.0278	0.0342	35.1
211	1.26	1.12	1.81	0.00028	0.0280	0.0167	75.4
212	2.50	2.32	1.01	0.002683	0.0283	0.0518	48.3
213	2.36	2.31	0.91	0.003088	0.0284	0.0556	42.4
214	4.66	4.14	1.87	0.00366	0.0284	0.0605	77.0
215	0.87	0.84	0.69	0.000594	0.0290	0.0244	35.7
216	1.18	1.11	1.36	0.000411	0.0292	0.0203	58.1
217	1.69	1.70	0.56	0.0033	0.0293	0.0574	29.4
218	0.61	0.61	1.08	0.00017	0.0295	0.0130	46.7
219	0.87	0.83	1.15	0.00029	0.0299	0.0170	51.0

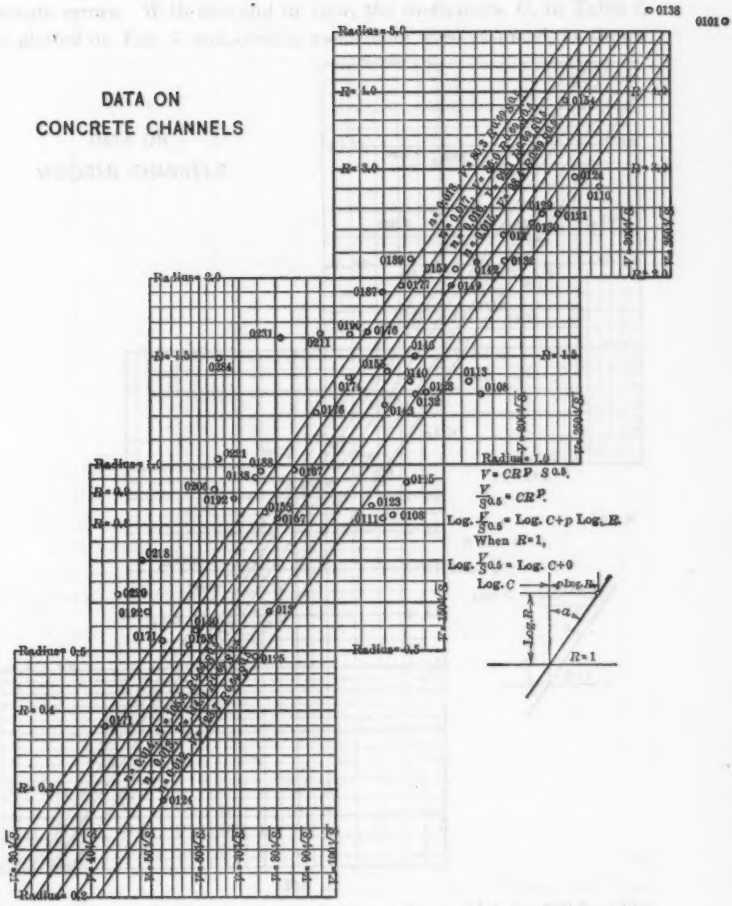
TABLE 1.—(*Continued.*)

EARTH CANALS, TYPE V.—(*Continued.*)

No. of experiment.	Measured velocity.	Velocity by formula: $V = 44.4\ R^{0.69}\ S^{0.5}$.	Radius.	Slope.	n.	$S^{0.5}$.	$C\ Rp$.
220	1.09	1.13	0.83	0.00084	0.0308	0.0290	37.6
221	0.91	0.92	0.88	0.0005165	0.0300	0.0227	40.1
222	1.73	1.73	1.88	0.000634	0.0315	0.0252	68.6
223	1.60	1.66	1.22	0.0010647	0.0318	0.0326	49.0
224	1.37	1.62	1.32	0.0009095	0.0360	0.0301	45.3
225	0.88	1.01	0.49	0.0014	0.0319	0.0374	23.5
226	1.10	1.00	0.48	0.0014	0.0320	0.0374	29.4
227	1.16	1.18	2.06	0.00026	0.0321	0.0161	72.0
228	1.03	1.07	1.07	0.000533	0.0324	0.0231	44.6
229	0.43	1.53	0.52	0.000353	0.0329	0.0188	22.9
230	1.20	1.44	0.73	0.001629	0.0346	0.0403	29.8
231	1.12	1.31	0.93	0.00093	0.0349	0.0305	36.7
232	0.84	0.98	1.32	0.00033	0.0352	0.0182	46.1
233	1.18	1.42	1.32	0.0006982	0.0364	0.0264	44.7
234	1.44	1.67	1.87	0.00060	0.0371	0.0245	58.8
235	0.81	0.98	1.80	0.000217	0.0373	0.0147	55.0
236	1.18	1.47	1.39	0.000695	0.0379	0.0264	44.7
237	0.70	0.92	1.06	0.000398	0.0381	0.0199	35.0
238	0.44	0.63	0.77	0.00029	0.0393	0.0170	25.9
239	1.44	1.84	1.84	0.000746	0.0397	0.0273	52.7
240	0.45	0.70	0.40	0.000919	0.0399	0.0303	14.8
241	0.35	0.54	0.56	0.0003367	0.0403	0.0183	19.1
242	1.52	2.10	1.62	0.00115	0.0424	0.0339	44.8
243	0.65	1.05	0.68	0.00095	0.0436	0.0308	21.1
244	1.45	1.99	1.8	0.000894	0.0436	0.0299	48.5
245	0.40	0.54	3.90	0.000023	0.0461	0.0048	83.3
246	0.65	1.08	1.74	0.00028	0.0499	0.0167	38.9
247	0.42	0.87	0.48	0.00107	0.0519	0.0327	12.8
248	0.37	0.77	0.58	0.00064	0.0529	0.0253	14.6
249	0.32	0.73	0.52	0.00067	0.0544	0.0259	12.3

TABLE 2.

Type.		n.	C.	α.	Tan. α.	Weights.	Tan. α as weighted.
Concrete.	I.....	0.012	128.0	32°30′	0.6371	0.7	0.44597
	II.....	0.013	113.5	34°15′	0.6809	0.8	0.54472
	III.....	0.014	107.5	33° 0′	0.6494	0.8	0.51952
	IV.....	0.015	99.0	33°30′	0.6619	0.7	0.46333
	V.....	0.016	91.5	35° 0′	0.7002	0.8	0.56016
	VI.....	0.017	85.0	35°40′	0.7177	0.8	0.57416
	VII.....	0.018	80.0	34° 0′	0.6745	0.7	0.47215
Wood.....	I.....	0.012	124.0	34° 0′	0.6745	0.5	0.33725
	II.....	0.013	114.0	34°30′	0.6873	0.6	0.41238
	III.....	0.014	106.0	35°30′	0.7133	0.8	0.57064
	IV.....	0.015	99.5	35°30′	0.7133	0.7	0.49931
	V.....	0.016	91.0	35°40′	0.7177	0.7	0.50239
Metal.....	I.....	0.011	137.0	32°15′	0.6309	0.4	0.25236
	II.....	0.015	93.5	33°50′	0.6708	0.4	0.26812
	III.....	0.022	.....	.....	.....	.....	.....
Earth.....	I.....	0.016	90.5	34° 0′	0.6745	0.8	0.53960
	II.....	0.020	70.3	35°20′	0.7089	0.7	0.49623
	III.....	0.0225	61.2	35°40′	0.7177	0.8	0.57416
	IV.....	0.025	52.8	36°55′	0.7513	0.7	0.52591
	V.....	0.030	44.4	37°30′	0.7673	0.6	0.46038
Means.....					0.692	.....	0.692
Sum.....					.....	13.0	.....


DATA ON
CONCRETE CHANNELS
Radius= 5.0
Radius= 2.0
Radius= 1.0
Radius= 0.5
R= 1.0
R= 3.0
R= 1.5
R= 0.4
R= 0.3
S= 0.0016
S= 0.0025
S= 0.0036
S= 0.0049
S= 0.0064
S= 0.0081
S= 0.0100
S= 0.0121
S= 0.0144
S= 0.0169
S= 0.0196
S= 0.0225
S= 0.0256
S= 0.0289
S= 0.0324
S= 0.0361
S= 0.0400
S= 0.0441
S= 0.0484
S= 0.0529
S= 0.0576
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S= 0.0841
S= 0.0900
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S= 0.1024
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S= 0.1156
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S= 0.1296
S= 0.1369
S= 0.1444
S= 0.1521
S= 0.1600
S= 0.1681
S= 0.1764
S= 0.1849
S= 0.1936
S= 0.2025
S= 0.2116
S= 0.2209
S= 0.2304
S= 0.2401
S= 0.2500
S= 0.2601
S= 0.2704
S= 0.2809
S= 0.2916
S= 0.3025
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S= 0.3364
S= 0.3481
S= 0.3600
S= 0.3721
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S= 0.3969
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S= 0.4225
S= 0.4356
S= 0.4489
S= 0.4624
S= 0.4761
S= 0.4900
S= 0.5041
S= 0.5184
S= 0.5329
S= 0.5476
S= 0.5625
S= 0.5776
S= 0.5929
S= 0.6084
S= 0.6241
S= 0.6400
S= 0.6561
S= 0.6724
S= 0.6889
S= 0.7056
S= 0.7225
S= 0.7396
S= 0.7569
S= 0.7744
S= 0.7921
S= 0.8100
S= 0.8281
S= 0.8464
S= 0.8649
S= 0.8836
S= 0.9025
S= 0.9216
S= 0.9409
S= 0.9604
S= 0.9801
S= 1.0000
V = CRP S 0.5.
V/S0.5 = CRP.
Log. V/S0.5 = Log. C+p Log. R.
When R=1,
Log. V/S0.5 = Log. C+0
Log. C
Log. R
R=1

0130

Table 2, then, gives a series of disconnected formulas for the flow of water in various types of channels, whereas one formula, or at least a uniformly varying series, is desirable, both for simplicity and to eliminate errors. With this end in view, the coefficients, C, in Table 2, were plotted on Fig. 3, and certain coefficients were assumed, as shown

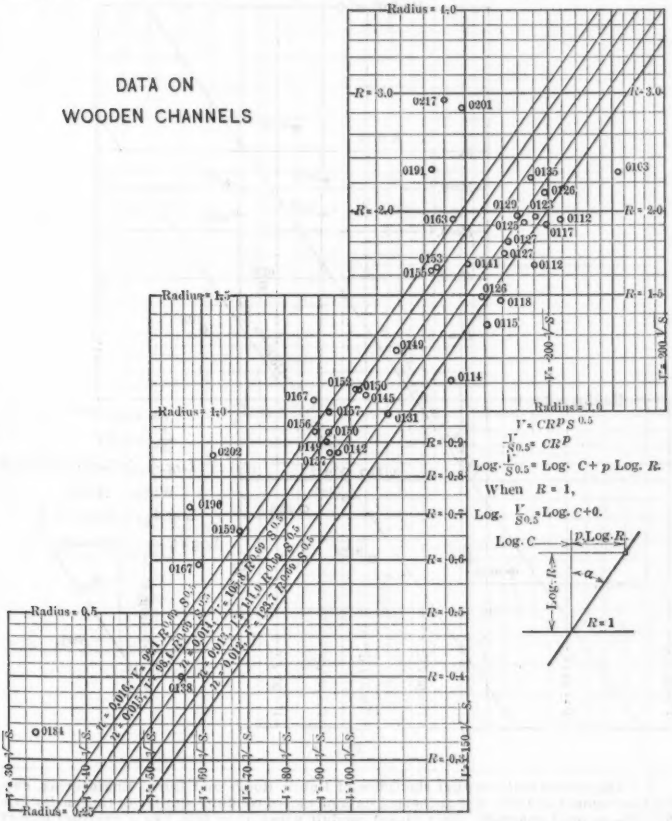


FIG. 1.

on that figure, varying on a smooth curve from 44.4 to 123.7. The exponents of R (the tangents of α in Table 2), plotted on Fig. 4, do not give so uniform a curve. The value of p might be taken as almost anything between $\frac{2}{3}$ and $\frac{3}{4}$, with little error, although the

drawing shows that its value increases with that of n. It is thought that this variation might be eliminated by using a different exponent for the slope, but the writer does not know how to find that.*

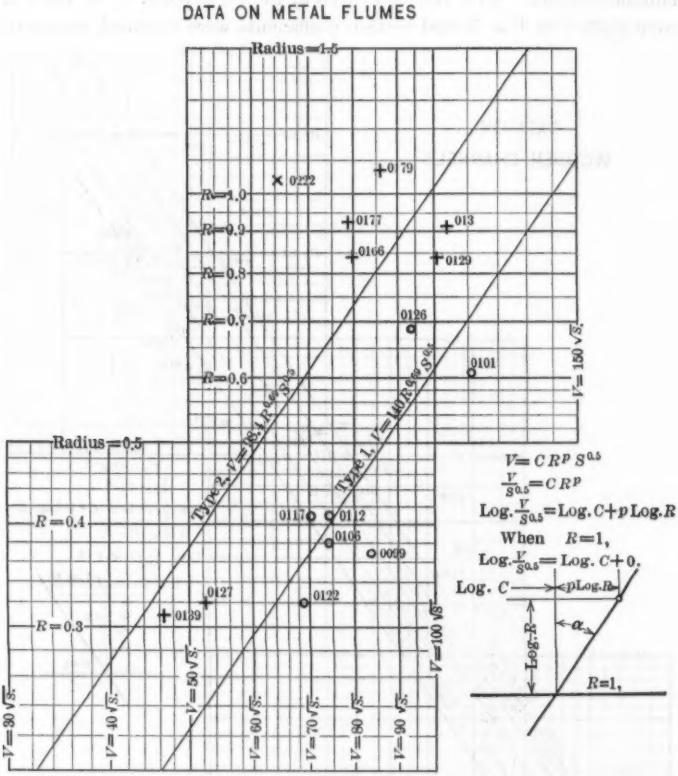


FIG. 2.

* The writer believes that the Office of Public Roads and Rural Engineering, during the summer of 1915, was gathering data on closed conduits similar to that in *Bulletin No. 194* on open channels. On a closed conduit which runs full, like a pressure pipe or siphon, the radius for a given diameter is constant, and does not vary with different velocities and slopes, the slopes being measured by the loss of head. The coefficient of roughness, of course, is a constant for any one conduit, so that the expression, $C\ R^p$, is constant. When those data are published, therefore, the ratings on each pipe can be plotted in the form, $V = (C\ R^p)\ S^q$ and the exponent of S can be determined. The average of these can then probably be applied to all the data, both in that bulletin and in this paper, and a new formula can be deduced, which, with the proper coefficient, will apply to all conditions of flow for either open or closed channels.

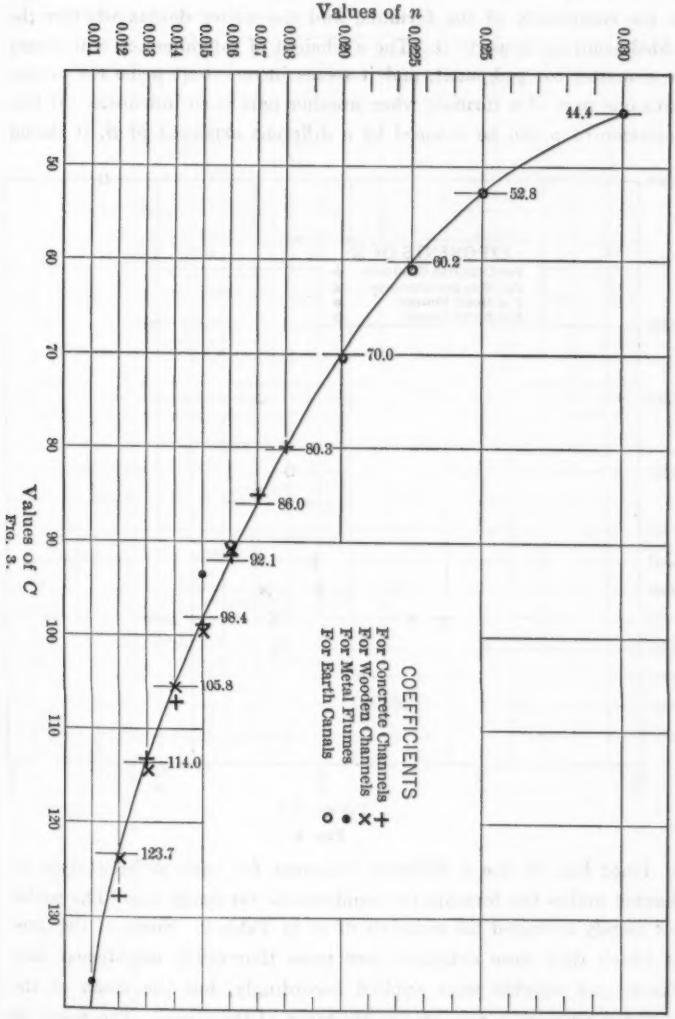

Values of n
0.011
0.012
0.013
0.014
0.015
0.016
0.017
0.018
0.020
0.0225
0.025
0.030
Values of C
50
60
70
80
90
100
110
120
130
44.4
52.8
60.2
70.0
80.3
86.0
92.1
98.4
105.8
114.0
123.7
COEFFICIENTS
For Concrete Channels +
For Wooden Channels ×
For Metal Flumes •
For Earth Canals o

FIG. 3.

Messrs. Johnston and Goodrich recognized this variation, and used different exponents of R for different types of channels. This adds to the complexity of the formula, and the writer doubts whether the added accuracy is worth it. The coefficient of roughness, n, will always be a matter for judgment, and it seems inconsistent to be too precise with one part of a formula when another part is so indefinite. If this variation of p can be reduced by a different exponent of S, it should

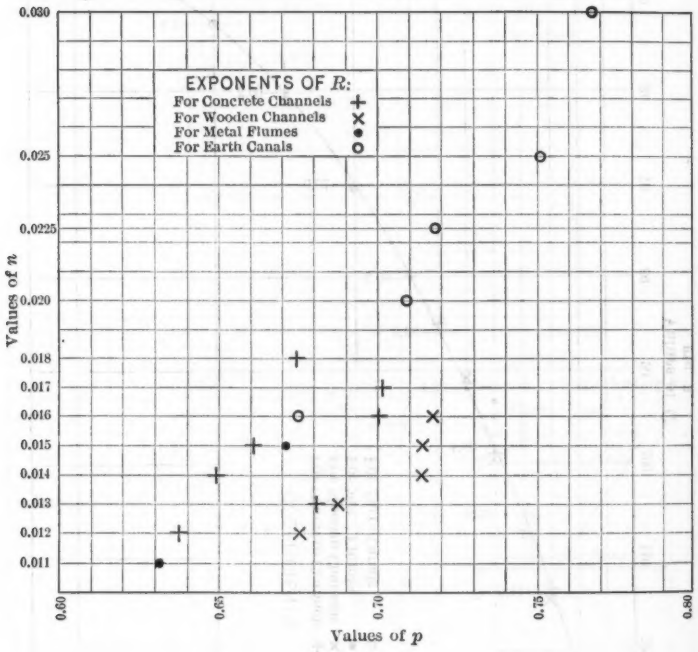
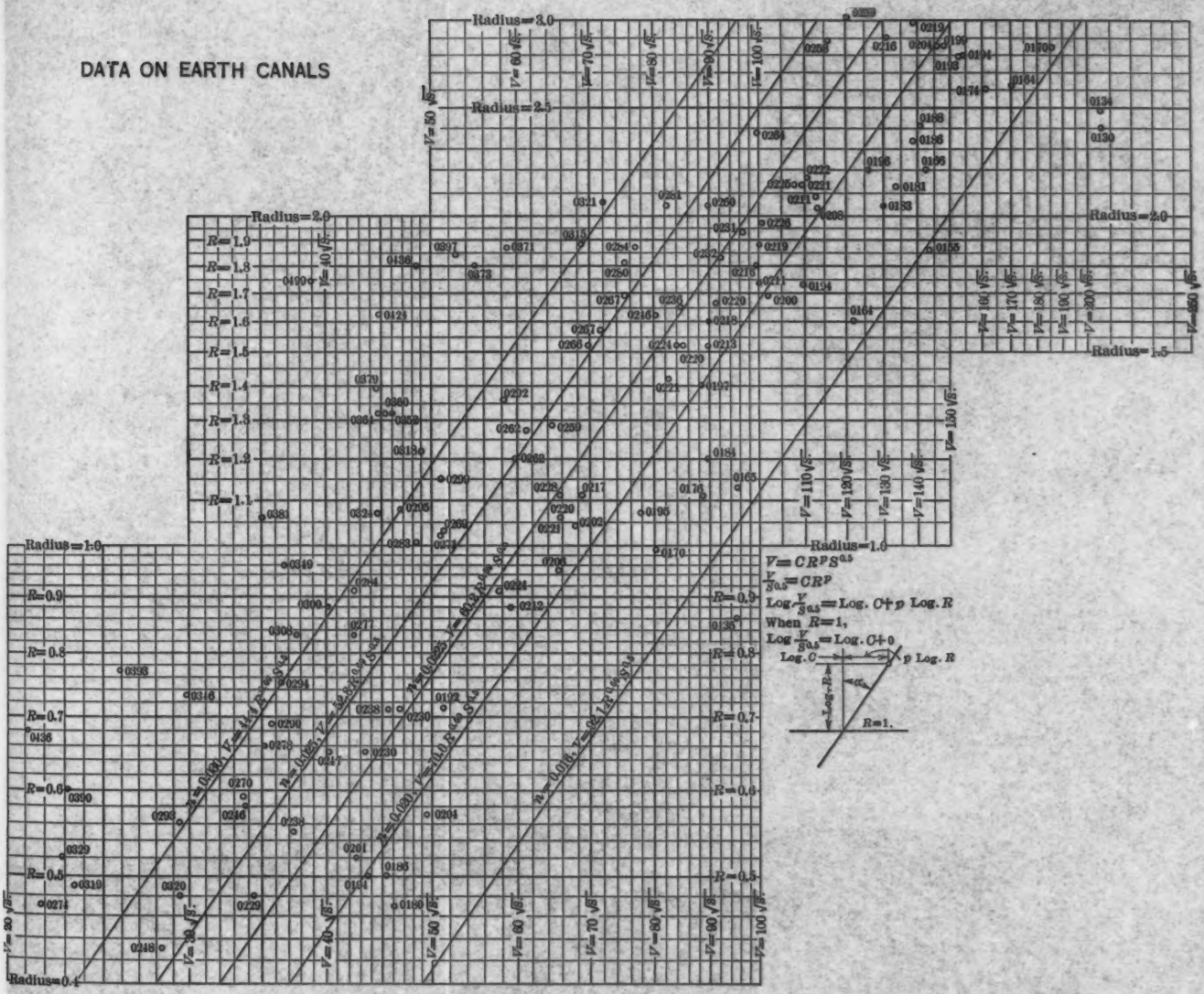


FIG. 4.

be done; but, to use a different exponent for each different type of channel makes the formula too cumbersome for ready use. The writer has merely averaged the tangents of α, in Table 2. Some of the lines by which they were obtained were more thoroughly established than others, and weights were applied accordingly, but the mean of the weighted tangents is the same as the mean of the others. The resulting formulas are represented by the diagonal lines on Plates LIX and LX

DATA ON EARTH CANALS



$V = CR^p S^{0.5}$

$\frac{V}{S^{0.5}} = CR^p$

$\text{Log.} \frac{V}{S^{0.5}} = \text{Log. } C + p \text{ Log. } R$

When $R = 1$,

$\text{Log} \frac{V}{S^{0.5}} = \text{Log. } C + 0$

and Figs. 1 and 2, and seem to fit the data fairly well. These formulas were used also in computing the theoretical velocities in the third column of Table 1, for ready comparison with the actual measured velocities.

Table 3, the 0.69th powers of radii, is added in order to aid in using or discussing these formulas.

TABLE 3.—0.69TH POWERS OF RADII.

R.	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.1	0.204	0.216	0.229	0.241	0.254	0.266	0.279	0.292	0.304	0.317
0.2	0.329	0.340	0.350	0.361	0.372	0.382	0.393	0.404	0.414	0.425
0.3	0.436	0.445	0.455	0.464	0.474	0.484	0.493	0.503	0.512	0.522
0.4	0.531	0.540	0.549	0.558	0.567	0.576	0.584	0.593	0.602	0.611
0.5	0.620	0.628	0.636	0.645	0.653	0.661	0.670	0.678	0.686	0.695
0.6	0.703	0.711	0.719	0.727	0.734	0.742	0.750	0.758	0.766	0.774
0.7	0.782	0.789	0.797	0.804	0.812	0.820	0.827	0.835	0.842	0.850
0.8	0.857	0.864	0.872	0.879	0.886	0.893	0.901	0.908	0.915	0.923
0.9	0.930	0.937	0.944	0.951	0.958	0.965	0.972	0.979	0.986	0.993
1.0	1.000	1.007	1.014	1.020	1.027	1.034	1.041	1.047	1.054	1.061
1.1	1.068	1.074	1.081	1.088	1.094	1.101	1.108	1.114	1.121	1.127
1.2	1.134	1.140	1.147	1.153	1.160	1.166	1.173	1.179	1.186	1.192
1.3	1.198	1.205	1.211	1.217	1.223	1.230	1.236	1.242	1.249	1.255
1.4	1.261	1.267	1.274	1.280	1.286	1.292	1.298	1.304	1.310	1.317
1.5	1.323	1.329	1.334	1.340	1.347	1.353	1.359	1.365	1.371	1.377
1.6	1.383	1.389	1.395	1.401	1.407	1.412	1.418	1.424	1.430	1.436
1.7	1.442	1.448	1.454	1.460	1.465	1.471	1.477	1.483	1.488	1.494
1.8	1.500	1.506	1.512	1.517	1.523	1.529	1.534	1.540	1.546	1.551
1.9	1.557	1.563	1.568	1.574	1.580	1.585	1.591	1.596	1.602	1.608
2.0	1.613	1.619	1.624	1.630	1.635	1.641	1.646	1.652	1.657	1.663
2.1	1.668	1.674	1.679	1.685	1.690	1.696	1.701	1.707	1.712	1.717
2.2	1.723	1.728	1.734	1.739	1.744	1.750	1.755	1.760	1.766	1.771
2.3	1.777	1.782	1.787	1.792	1.798	1.803	1.808	1.814	1.819	1.824
2.4	1.829	1.835	1.840	1.845	1.850	1.856	1.861	1.866	1.871	1.876
2.5	1.882	1.887	1.892	1.897	1.902	1.907	1.913	1.918	1.923	1.928
2.6	1.933	1.938	1.944	1.949	1.954	1.959	1.964	1.969	1.974	1.979
2.7	1.984	1.989	1.995	2.000	2.005	2.010	2.015	2.020	2.025	2.030
2.8	2.035	2.040	2.045	2.050	2.055	2.060	2.065	2.070	2.075	2.080
2.9	2.085	2.090	2.095	2.100	2.104	2.109	2.114	2.119	2.124	2.129
3.0	2.134	2.129	2.144	2.149	2.154	2.158	2.163	2.168	2.173	2.178
3.1	2.183	2.188	2.192	2.197	2.202	2.207	2.212	2.217	2.222	2.226
3.2	2.231	2.236	2.241	2.246	2.250	2.255	2.260	2.265	2.270	2.274
3.3	2.279	2.284	2.289	2.293	2.298	2.303	2.308	2.312	2.317	2.322
3.4	2.326	2.331	2.336	2.341	2.345	2.350	2.355	2.359	2.364	2.369
3.5	2.374	2.378	2.383	2.388	2.392	2.397	2.402	2.406	2.411	2.416
3.6	2.420	2.425	2.429	2.434	2.439	2.443	2.448	2.453	2.457	2.462
3.7	2.466	2.471	2.475	2.480	2.485	2.489	2.494	2.498	2.503	2.508
3.8	2.512	2.517	2.521	2.526	2.530	2.535	2.539	2.544	2.548	2.553
3.9	2.558	2.562	2.567	2.571	2.576	2.580	2.585	2.589	2.594	2.598
4.0	2.603	.....	.....	.....	.....	.....	.....	.....	.....	.....

DISCUSSION

Mr. Muckleston.

H. B. MUCKLESTON,* M. AM. SOC. C. E. (by letter).—The principal reason for the existence of discharge formulas is for purposes of design, so that the engineer can be reasonably sure that a canal or conduit will discharge at least the quantity required by its service. If this condition is satisfied, the designer has done all he can, and may rest content.

There have been a great many such formulas devised, ranging in complexity from the very simple Chezy expression through the awful Dubuat and Kutter equations, and back to the comparatively easy experimental formulas which seem to hold the stage at present. As the author very well states, something in the way of a coefficient to allow for varying degrees of roughness has to be assumed in any event, and it seems illogical to torture a formula with odd decimal exponents or complicated coefficients, when round figures would give results quite as close to the probable truth.

The author states that the value of the exponent p might be chosen as almost anything between $\frac{2}{3}$ and $\frac{3}{4}$, with little error. The writer pleads for a value of $\frac{2}{3}$, for the sole reason that the expression can then be solved with a slide-rule, and, given a good memory and a duplex rule, the designer needs no tables, diagrams, or other aids. The writer also confesses to a decided preference for Manning's formula, because it retains the familiar symbol, n, for the roughness factor.

It has been the writer's experience that, for earthen channels at any rate, there is no constancy about any roughness factor. In one series of experimental determinations, at the same site, the value of n (Kutter) varied from 0.018 at a low stage to 0.0248 at nearly full supply. In another series, apparently made under similar conditions, but at a different site, the variations were in the opposite direction. Both series of experiments were made on a canal which had been in service 9 years.

The writer will admit that, in the case of lined channels, the designer can predict with somewhat more certainty that the assumed conditions of roughness will obtain in the finished structure, but, even then, variations in the roughness factor may be expected with varying stages.

The author makes the statement that "water runs in a canal by merely sliding down hill". It would be nearer the truth to say that it flows by falling over "itself". The early hydraulicians worked on the very convenient assumption that water flowed in a sort of series

* Calgary, Alberta, Canada.

of concentric envelopes, each of them influencing and being influenced by its neighbor on each side. It was this theory that was responsible for the intangible dimension, R, which appears directly or indirectly in every formula since Chezy's. This dimension may have a physical existence in the case of a round pipe, and possibly, as a consequence, experimental results on such pipes seem to be more consistent among themselves, but for conduits of other shapes it would appear that a further factor is needed to allow for the shapes, or to correct for the use of R, a non-existent dimension. As far as the writer is aware, the only formula which contains such a factor is attributed to Professor Claxton Fidler, and is as follows: Mr. Muckleston.

$$\text{Log. } S = x\,(m + \text{log. } V - \frac{2}{3}\,\text{log. } R) - 6.00,$$

in which m is a constant depending on shape, and x is a constant of roughness. He gives a demonstration of this formula,* with values of the coefficients, m and x, and also a modification of the formula for use with earthen channels.

Finally, it appears to the writer that, if experiments prove anything at all, it is that water in open and closed conduits does not follow the same law. In an open conduit, the slope is constant and the velocity depends on the hydraulic mean radius; in the closed conduit, the radius is constant and the velocity depends on the slope or hydraulic grade. If we take pipe experiments for pipe formulas and open-channel experiments for open-channel formulas, we shall be on surer ground, and our formulas should be more logical.

HAROLD F. GRAY,† JUN. AM. SOC. C. E.‡ (by letter).—The question of a relatively simple, yet reasonably accurate, formula for the flow of water in open channels is one which has engaged the attention of hydraulic engineers for a long time. Probably no formula has been so widely used as that of Ganguillet and Kutter, but this has always been open to considerable objection on account of its complexity, as well as certain inaccuracies. Bazin's formula is an improvement, in that it is simpler, but it is doubtful if it presents much greater accuracy than that of Kutter. It has not been used extensively by American engineers. The possibility of developing a simpler and satisfactory formula of the exponential type has been considered, particularly since the successful application of a formula of this type to the flow of water in pipes has accustomed engineers to its use. Mr. Gray.

The writer has been interested in this problem, and three years ago attempted to make comparisons between exponential formulas and those of Kutter and Bazin. From the exponential formula§ used by the writer, he developed a diagram, Fig. 5, for its use. This diagram was

* "Calculations in Hydraulic Engineering."
† San José, Cal.
‡ Now Assoc. M. Am. Soc. C. E.
§ "American Civil Engineers' Pocket Book", 2d Ed., p. 858.

Mr. Gray.

used, in connection with diagrams for the Kutter and Bazin formulas, in determining what values of C were necessary, in order to obtain, for identical channels, the same velocity with the exponential formula as with the formulas of Kutter and of Bazin. Table 4 shows the variations in the coefficient necessary to bring the exponential formula into conformity with the Bazin formula. The tabular comparison for the Kutter formula has been lost.

TABLE 4.—APPROXIMATE VALUES OF C FOR USE WITH THE DIAGRAM, FIG. 5, FOR ESTIMATING THE FLOW OF WATER IN OPEN CHANNELS BY THE EXPONENTIAL FORMULA: $V = C\,R^{0.67}\,S^{0.54}$

I.—VERY SMOOTH CEMENT: PLANED BOARDS.

R	Slope.					
	0.0001	0.0004	0.001	0.003	0.01	0.05
0.5	215	210	205	200	195	185
1	205	200	196	185	175	165
2	190	180	170	165	160	150
3	180	170	160	155	150	140
5	165	155	150	140	135	130
7	155	145	140	135	125	120
10	145	135	130	125	120	115

II.—SMOOTH BOARDS, BRICK, CONCRETE, SEWER PIPE.

R	Slope.					
	0.0001	0.0004	0.001	0.003	0.01	0.05
0.5	175	170	165	160	155	145
1	170	165	160	155	150	140
2	165	160	155	150	140	130
3	160	150	145	140	135	125
5	150	145	140	135	130	120
7	145	135	135	130	120	110
10	140	130	125	120	115	105

III.—SMOOTH BUT DIRTY BRICK OR CONCRETE.

0.5	150	145	140	135	125	120
1	150	140	135	130	120	115
2	150	140	135	125	120	115
3	145	135	130	125	115	110
5	140	130	125	120	115	110
7	135	125	120	115	110	105
10	130	120	115	110	105	100

IV.—ASHLAR MASONRY.

0.5	115	110	105	100	100	100
1	120	110	110	105	100	100
2	120	115	115	110	100	100
3	125	120	115	110	105	100
5	125	115	110	110	105	100
7	120	115	110	110	100	100
10	115	110	110	105	100	100

V.—EARTH CANALS IN VERY GOOD CONDITION.

0.5	80	75	70	65	65	65
1	85	80	75	70	70	70
2	90	85	80	75	75	75
3	95	90	85	80	80	80
5	100	90	90	85	85	80
7	100	95	90	85	85	80
10	100	95	95	90	85	85

VI.—EARTH CANALS IN ORDINARY CONDITION.

0.5	65	60	55	50	50	50
1	70	65	60	55	55	55
2	70	65	65	60	60	60
3	75	70	70	65	65	65
5	80	75	75	70	70	65
7	85	80	75	70	70	65
10	85	80	80	75	70	65

VII.—EARTH CANALS EXCEPTIONALLY ROUGH.

0.5	50	45	40	35	30	25
1	55	50	45	40	40	35
2	60	55	50	50	50	45
3	65	60	55	55	55	50
5	70	65	60	60	60	55
7	75	65	65	65	60	55
10	75	70	65	65	60	55

In Table 4, Group I, for very smooth cement and planed boards, shows coefficients agreeing with Γ equals 0.109 in the Bazin formula;

Mr. Gray.

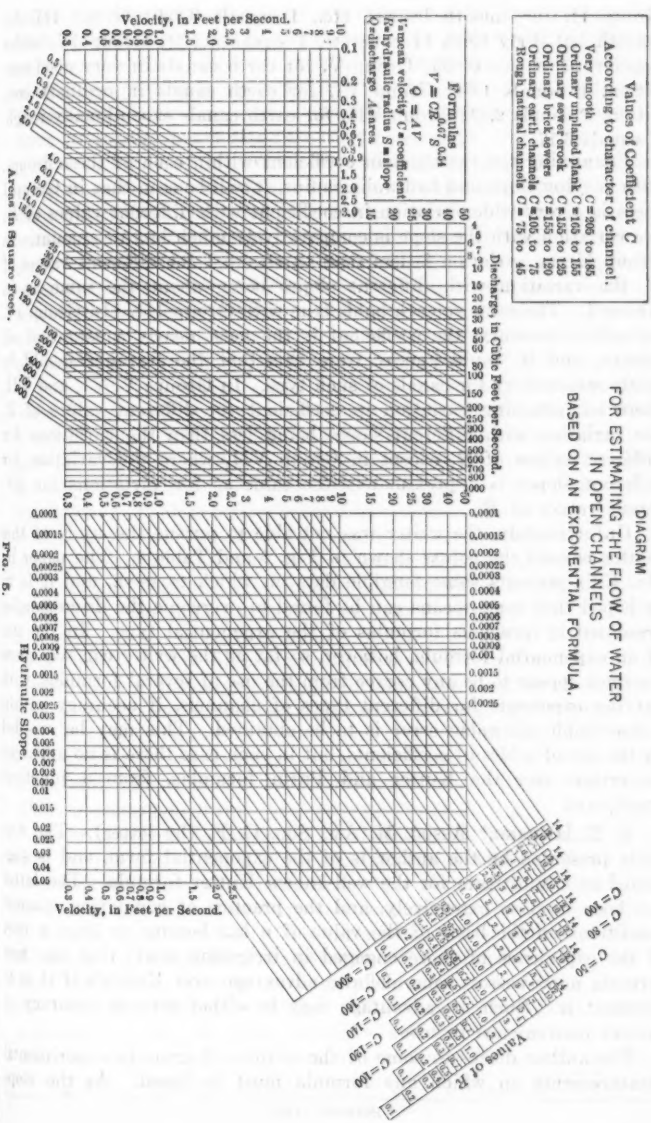


DIAGRAM
FOR ESTIMATING THE FLOW OF WATER
IN OPEN CHANNELS
BASED ON AN EXPONENTIAL FORMULA.
Values of Coefficient C
According to character of channel.
Very smooth C = 165 to 185
Ordinary unplaned plank C = 145 to 155
Ordinary planed plank C = 135 to 145
Ordinary brick sewers C = 125 to 130
Ordinary earth channels C = 105 to 75
Rough, natural channels C = 75 to 45
Formulas
V = CR^0.67 S^0.54
Q = AV
V = mean velocity C = coefficient
R = hydraulic radius S = slope
Q = discharge A = area
Velocity, in Feet per Second.
Discharge, in Cubic Feet per Second.
Areas in Square Feet.
Hydraulic Slope.
Velocity, in Feet per Second.
Values of R
C = 100
C = 120
C = 130
C = 140
C = 150
C = 160
C = 180
C = 200

FIG. 5.

Mr. Gray. Group II, for smooth boards, etc., Γ equals 0.290; Group III, for smooth but dirty brick or concrete, Γ equals 0.500; Group IV, ashlar masonry, Γ equals 0.833; Group V, for earth canals in very good condition, Γ equals 1.54; Group VI, for earth canals in ordinary condition, Γ equals 2.35; Group VII, for earth canals exceptionally rough, Γ equals 3.17.

Examining the variation in coefficient within each of these groups, with varying slope and hydraulic radius, it will be seen that in Group I there is a very wide variation in coefficient with different values of R; the variation with the slope is comparatively much less in amount. In Group II the variation is less than in Group I for different values of R, the variation with different slopes being about the same as in Group I. The same remark holds true with Group III. In Group IV, for ashlar masonry, the coefficient is the most nearly constant of all groups, and it would appear from this that the formula should be fairly satisfactory for canals of such type. In Groups V, VI, and VII, there is increasing variation in coefficient for different values of R, the variation with different slopes being less than the variations for different values of R, except in Group VII, where the variation for different slopes is approximately the same as for variations for different values of R.

Unfortunately, the writer was not able to pursue the study of this matter beyond the points shown in Fig. 5 and Table 4. The paper by Mr. Ellis presents some tangible data on which to work, and it is to be hoped that some means can be found to overcome the discrepancies presented by previous formulas of the exponential type. In the use of an exponential formula, however, so far as the writer can see, there does not appear to be any escape from the use of several formulas, with varying exponents for different types of channels, if accuracy within a reasonable allowable error is to be obtained. This may be avoided by the use of tables of coefficients, but in such case there is no apparent advantage over the Kutter and Bazin formulas, from a practical standpoint.

Mr. Harding. S. T. HARDING,* ASSOC. M. AM. SOC. C. E. (by letter).—The formula presented by the author is of the exponential form, and is suggested as a substitute for the well-known Kutter formula. The latter has been used so extensively, and the practice of comparing channel conditions on the basis of the value of n has become so large a part of the experience of those engaged in irrigation work, that any new formula must possess some definite advantage over Kutter's if it is to supplant it. Such an advantage may be either greater accuracy or greater convenience in use.

The author discusses some of the sources of error in experimental measurements on which any formula must be based. As the slope

* Berkeley, Cal.

Mr. Harding.

enters the result at only the 0.50th power in his formula, the error in the resulting velocity in the case cited by him would be only 3.7% on a canal with a slope as small as 0.0001. The writer made about 45 of the experiments given in *Bulletin No. 194*, on which the paper is based; and the levels were checked on these, so as to reduce the error in slope to much less than that percentage.

The accuracy of current-meter gaugings in large canals is necessarily uncertain, as the field conditions do not permit of the use of other methods which would give absolute checks. The flow and cross-section are more uniform than in many river gauging stations, and greater average accuracy should be attained. In connection with the investigations mentioned in *Bulletin No. 194*, experiments on the use of current meters in different methods of gauging were made at the same time, the particulars of which have been published.* These experiments give comparisons of the consistency of different methods of using current meters, rather than comparisons of any actual accuracy. They indicate that a consistency within 1 or 2% can be obtained. Current meters are similar to other engineering instruments; they may be used so as to give accurate results, or they may be handled so as to produce results only approximately correct, just as a transit may be handled so as to close a traverse within a minute of angle, or not within several minutes. The failure to secure accurate results is more commonly due to the conditions of use than to the fault of the instrument.

With any formula, greater accuracy cannot be expected than that with which the empirical coefficient can be selected. With Kutter's formula, it is not usual to attempt to select the value of n within about 10 per cent. A 10% difference in the value of n in that formula gives a difference in discharge for a given cross-section of from 12 to 15%; and, to carry the same discharge on the same grade, requires about 8% increase in water cross-sectional area. With the large number of experiments now available, it should usually be possible to select coefficients and exponents approximating these limits for any formula which is correct in form.

Some evidence is presented in *Bulletin No. 194* tending to show that the value of n for a given channel varies with the depth of flow or the value of the hydraulic radius. This would indicate that Kutter's formula is not correct in form and does not represent exactly the law of flow in open channels. The variations, however, are not uniform. Johnston and Goodrich have presented a diagram† showing the variation of Kutter's n with the hydraulic radius, n becoming greater as the hydraulic radius diminishes. The values of n given are those generally adopted for earth channels, and are higher than those for artificial sections such as in wood or concrete. The actual experiments

* *Journal of Agricultural Research*, Vol. V, No. 6, November 8th, 1915.
† *Engineering Record*, Vol. 64, November 4th, 1911.

Mr. Harding.

on which this diagram is based have not been published, as far as the writer is aware. It is difficult to see how dependable results on this point can be secured, except with channels of fixed and uniform cross-section and uniform surface. Such conditions are found in irrigation practice only in flumes or lined sections. In earth canals, the character of the wetted perimeter usually varies with the depth; grass and weeds may be submerged at high stages; the sides may have silt deposits; and the bottom may be clean or possibly somewhat gravelly. As the depth—and therefore value of the hydraulic radius—decreases, the value of n may easily vary, due to the greater relative importance of the character of the bottom of the canal. In *Bulletin No. 194* of the Colorado Agricultural Experiment Station, some data—also inconclusive—suggest an increase in the value of n with the discharge. In view of these conflicting and uncertain results, it seems safe to consider Kutter's formula as being sufficiently near to the theoretically correct form for all practical purposes.

Speaking broadly, Kutter's formula has no disadvantage, with reference to convenience in use, when compared with the various exponential formulas. With so many tables and curves available for the solution of Kutter's formula, actual computation in any case is unnecessary and unwarranted. The most important feature—relating to convenience in use—is the directness with which an engineer's experience in the selection of the empirical coefficient—such as n in Kutter's formula—can aid him in ascertaining the resulting velocity for any values of R and S. This can be done, at least at present for American engineers, most quickly and directly with Kutter's formula.

Some of the formulas mentioned by the author, and based on present knowledge to a sufficient extent to warrant consideration, are Kutter's, Manning's and his own. In addition, there are Bazin's new or 1897 formula,

$$V = \frac{157.6}{1 + \frac{C}{\sqrt{R}}} \sqrt{R\,S},$$

and Williams' formula for open channels, $V = C\,R^{0.67}\,S^{0.54}$.

The value of C in Bazin's formula depends on the character of the wetted perimeter. Various values are given in different books, the details for earth canals being less explicit than for similar values for Kutter's n. The same may be said regarding the values of C in Williams' formula.

Owing to the general use of Kutter's n, in defining the character of the wetted perimeter, it is desirable to have some means of correlating n with the coefficients of these different formulas. Fig. 3 gives a basis for such correlation, and the author's plotting of this figure may be accepted. The range in the value of n, included in the different

mean points plotted, is somewhat large, but further detail probably would not change the curve materially. That curve follows the equation, $C = \frac{1.25}{n - 0.002}$, quite closely, as shown in Fig. 6. The author's formula may then be written Mr. Harding.

$$V = \frac{1.25}{n - 0.002} R^{0.69} S^{0.50}.$$

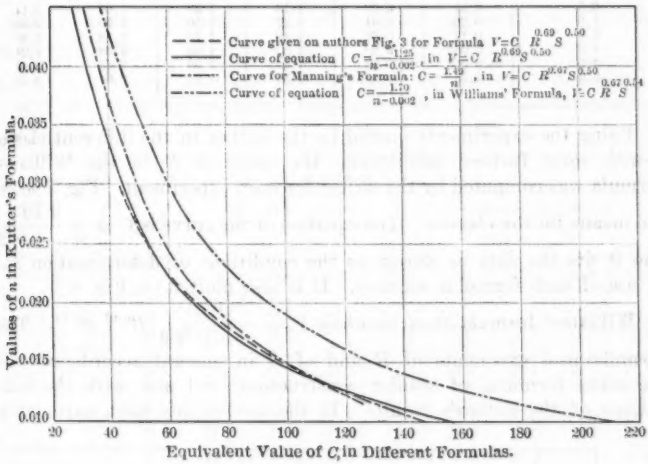


FIG. 6.

Kutter's n enters into Manning's formula directly. The author's and Bazin's new formula can be equated as follows:

$$\text{Author's formula: } \frac{V}{R^{0.69}\ S^{0.50}} = \frac{1.25}{n - 0.002}$$

$$\text{Bazin's formula: } \frac{V}{R^{0.69}\ S^{0.50}} = \frac{157.6}{1 + \frac{C}{\sqrt{R}}}\ \frac{1}{R^{0.19}}.$$

This reduces to the equation

$$C = 126.1\ (n - 0.002)\ R^{0.31} - R^{0.50}.$$

The solution of this equation for various values of n and R is given in Table 5.

The author suggests the need of using an exponent of S greater than 0.50. This is done in Williams' formula. The values of the exponent of S in the other formulas for closed conduits are so nearly the same as the 0.54 of Williams' formula that it appears safe to accept this value without elaborate investigation.

Mr. Harding.

TABLE 5.—VALUES OF C IN BAZIN'S (1897) FORMULA FOR DIFFERENT VALUES OF KUTTER'S n AND THE HYDRAULIC RADIUS.

Mean hydraulic radius, R.	VALUES OF C IN BAZIN'S (1897) FORMULA FOR THE FOLLOWING VALUES OF n IN KUTTER'S FORMULA:					
	0.012	0.015	0.020	0.0225	0.025	0.030
0.5	0.31	0.62	1.13	1.38	1.64	2.14
1.0	0.26	0.64	1.27	1.59	1.90	2.53
2.0	0.17	0.62	1.40	1.79	2.18	2.96
3.0	0.062	0.60	1.50	1.94	2.38	3.28
4.0	— 0.063	0.52	1.49	1.98	2.46	3.48
6.0	— 0.25	0.40	1.50	2.05	2.61	3.70

Using the experiments quoted by the author in the different classes —with some further subdivision—the value of C in the Williams' formula was computed by the writer for each experiment. Fig. 7 shows the means for the classes. The equation of the curve is $C = \frac{1.70}{n - 0.002}$, and it fits the data as closely as the conditions of determination and of use of such formulas warrant. It is also plotted on Fig. 6.

Williams' formula then becomes $V = \frac{1.70}{n - 0.002} R^{0.67} S^{0.54}$. This formula uses exponents of R and of S in general accordance with the other formulas of similar construction, and also with the indications of the author's results. If the use of any such exponential

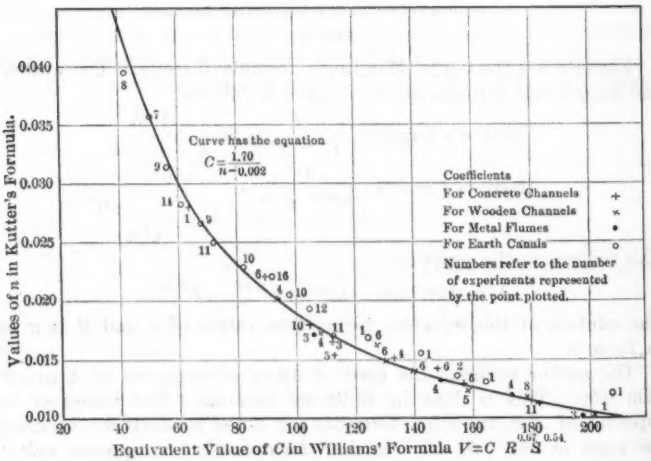


FIG. 7.

formula is considered preferable, it would appear that this one is the more logical. The computation of the 0.54th power of S requires the use of logarithms; the 0.50th power can be secured directly with the ordinary slide-rule. However, the exponent, 0.67, of R requires logarithms for the complete solution of the equation, so that this disadvantage is not of importance. Mr. Harding.

Fig. 6 and Table 5 enable one to interpret the value of Kutter's n into any of the other formulas given. Such a relation permits one to think in terms of Kutter's n, and compute by whichever formula may be preferred.

TABLE 6.—VALUE OF THE COEFFICIENT, C, IN THE FORMULA, $V = C\sqrt{RS}$, IN VARIOUS FORMULAS FOR DIFFERENT CONDITIONS.

Mean hydraulic radius.	n in Kutter's formula.	VALUE OF C.								
		Manning's formula, all slopes.	Ellis' formula, all slopes.	Kutter's Formula. For Slopes of:				Williams' Formula. For Slopes of:			
				0.001	0.0005	0.0002	0.0001	0.001	0.0005	0.0002	0.0001
0.5	0.012	110.4	109.3	111.5	109.5	106.2	101.0	114.6	111.4	107.5	104.5
	0.015	88.3	84.0	85.0	83.5	80.7	76.6	88.0	85.8	82.4	80.2
	0.020	66.2	60.7	59.6	58.9	56.4	53.6	63.6	61.9	59.7	58.0
	0.0225	58.8	53.3	51.9	51.2	49.0	46.8	55.8	54.3	52.3	50.9
	0.025	53.0	47.6	45.6	44.8	43.2	41.0	49.8	48.4	46.7	45.4
	0.030	44.2	39.0	36.4	35.9	34.6	32.9	49.0	39.7	38.3	37.3
1.0	0.012	124.2	125.0	127.4	126.5	123.9	120.1	129.0	125.4	121.0	117.6
	0.015	99.3	96.2	99.1	98.5	96.1	93.1	99.2	96.5	93.0	90.3
	0.020	74.5	69.4	71.5	70.0	69.2	66.9	71.6	69.6	67.2	65.3
	0.0225	66.2	61.0	62.5	61.9	60.4	58.6	62.8	61.1	58.9	57.3
	0.025	59.6	54.4	55.4	54.9	53.6	51.8	56.0	54.5	52.6	51.1
	0.030	49.7	44.6	44.9	44.5	43.5	42.1	46.0	44.7	43.1	41.9
2.0	0.012	139.7	142.6	141.8	141.4	140.3	138.6	145.1	141.0	135.9	132.3
	0.015	111.7	109.7	112.3	111.8	110.9	109.4	111.5	108.5	104.5	101.7
	0.020	83.8	79.2	82.9	82.5	81.8	80.6	80.5	78.3	75.4	73.4
	0.0225	74.5	69.6	73.2	72.9	72.2	71.2	70.7	68.7	66.2	64.4
	0.025	67.1	62.0	65.5	65.2	64.6	63.7	63.0	61.3	59.1	57.5
	0.030	55.9	51.0	54.0	53.8	53.2	52.5	51.7	50.3	48.5	47.2
3.0	0.012	149.6	154.0	149.5	149.3	149.0	148.8	155.4	151.1	145.7	141.7
	0.015	119.7	118.4	119.2	119.1	119.0	118.7	119.5	116.1	112.0	109.0
	0.020	89.8	85.6	89.2	88.9	89.0	88.8	86.3	83.8	80.9	78.7
	0.0225	79.8	75.1	79.2	79.2	79.0	78.8	75.7	73.5	70.9	69.0
	0.025	71.8	66.9	71.2	71.2	71.0	70.9	67.5	65.6	63.3	61.6
	0.030	59.9	55.0	59.2	59.2	59.1	58.9	55.4	53.8	51.9	50.5
4.0	0.012	157.3	162.6	154.3	154.4	154.9	155.6	163.2	158.8	153.1	148.9
	0.015	125.7	125.0	124.1	124.0	124.5	125.0	125.5	122.1	117.7	114.5
	0.020	94.4	90.3	93.5	93.6	93.9	94.4	90.6	88.2	85.0	82.7
	0.0225	83.8	79.5	83.3	83.4	83.7	84.2	79.5	77.3	74.6	72.5
	0.025	75.4	70.7	75.1	75.2	75.5	76.0	70.8	69.0	66.5	64.7
	0.030	62.9	58.1	62.9	63.0	63.2	63.6	58.1	55.6	54.6	53.1
6.0	0.012	168.5	175.6	160.4	160.8	162.3	164.5	175.0	170.2	164.0	159.5
	0.015	134.7	135.1	130.5	130.3	131.5	133.5	134.5	130.8	126.0	122.6
	0.020	101.0	97.6	99.0	99.5	100.6	102.2	97.1	94.5	91.0	88.5
	0.0225	89.8	85.7	88.7	89.1	90.1	91.6	85.2	82.9	79.9	77.7
	0.025	80.8	76.4	80.4	80.8	81.7	83.1	76.0	73.9	71.3	69.3
	0.030	67.4	62.7	67.9	68.2	69.0	70.2	62.4	60.6	58.4	56.9

Mr. Harding.

The values of the Chezy coefficient, C, for the different formulas, are given for certain values of R, n, and S in Table 6. Parts of this table are compiled from various sources and parts are computed for this purpose. These values are all reduced to the coefficient to be applied to $\sqrt{R\,S}$. The coefficient for Bazin's formula will depend somewhat on the basis of correlation of n and C. If that given in this discussion is used, the coefficients will be the same as for the Ellis formula, as they are derived from the same relation of n and C.

For the comparisons given, the Manning formula has coefficients higher than Kutter's for the larger values of n, but, for the lower values, the agreement is quite close; this is also indicated on Fig. 6. The variation seldom exceeds 10 per cent. The difference between the coefficients of the author's and Kutter's formulas varies with the different values of R, n, and S, the agreement usually being relatively good. On the steeper slopes, the coefficients from Williams' formula exceed those from Kutter's; on the flatter slopes the reverse is true.

For all formulas, the agreement is generally within the probable error in the prediction as to the actual nature of the wetted perimeter, or the selection of the empirical coefficient. From the standpoint of results, there appears to be little choice between these formulas. Kutter's has the advantage of established usage, and Williams' perhaps has the advantage of more simple form. Exponential formulas can be developed in indefinite number, having small variations in the exponents and coefficient. It is very questionable if such differences are sufficiently great to warrant the inconvenience of their use. There may be an opportunity to bring together all available experimental data in one general study of the best exponents to be used, but such data are voluminous, and the task would be laborious. The result would probably differ from Williams' formula to such a slight extent, if at all, as hardly to repay for the effort made.

Mr. Hazen.

ALLEN HAZEN,* M. AM. SOC. C. E. (by letter).—This paper is a discussion by the author of experimental results collected by another. The author is not responsible for the experiments. The results obtained by his discussion are consistent among themselves to an extraordinary degree. The first impression is that there must be unusual merit in his procedure; further study, however, discloses a fallacy in it.

The results are consistent because of the method of classification of the data that is used as a preliminary to determining the formula which applies to each separated part. For this classification of data the Kutter formula has been used as a basis. Each section of the results thus separated is that which will produce the same value of n in the Kutter formula. The author, in reality, has found a series of exponential formulas each producing approximately the same re-

* New York City.

Mr. Hazen.

sults as the Kutter formula for a certain range. The effect of the procedure, therefore, is to demonstrate certain equivalents of the Kutter formula. It serves no other purpose. Nothing is brought out which substantiates the accuracy of the coefficients and exponents which are suggested by the author.

The method used to produce this extraordinary result is as follows: There were 269 experiments on irrigation ditches.* The data for each experiment were arranged to show, among other things, the value of n in the Kutter formula for that experiment. This is as far as Mr. Scobey carried the study. The author has taken Mr. Scobey's results and classified them according to the values of n. For the experiment in each section thus classified the value of n is nearly constant. The experiments in each section are then used to determine an exponential formula. The formula thus ascertained accounts for these particular experiments; that is to say, for those experiments in which a certain value of n is found. The exponential formula thus found corresponds approximately with the Kutter formula, with that value of n, and for the range covered. It would be impossible for it to do anything else.

The results reached by the author might have been reached more readily and with greater accuracy by a direct analysis of the Kutter formula. For instance, referring to the coefficients of the Kutter formula as tabulated by Trautwine,† for a slope of 0.001, which may be taken as representative of these experiments, the increase in the coefficients in the table from 139 for $R = 1$ to 155 for $R = 2$ is as the 0.16th power of R. As R again comes into the calculation to the extent of the 0.50th power, by Kutter's formula, for these conditions the increase in velocity is as the 0.66th power of R. In the same way, for $n = 0.020$, this exponent becomes 0.73, and for $n = 0.030$ it becomes 0.75.

Comparing these values with those found by the author and plotted in Fig. 4, the correspondence is seen to be striking. In other words, the author has found an average exponent of 0.69 simply because 0.69 gives approximately the same average increase for these experiments as would be given by the Kutter formula. There is no other basis for this conclusion. There is no analysis of the experimental results that indicates what the true exponents are, and it may be questioned whether the data in the paper are sufficient to make possible any close determination of their values.

In the same way coefficients in the several formulas found by the author may be compared with the coefficients for the Kutter formula in the Trautwine table as in Table 7.

* Described in *Bulletin No. 194*, U. S. Department of Agriculture, by Frederick C. Scobey, Assoc. M. Am. Soc. C. E.

† "Civil Engineer's Pocket-book", 19th Ed., p. 568.

Mr. Hazen.

TABLE 7.

Values of n.	Ellis. Fig. 3.	Trautwine. $S = 0.001$ $R = 1$.
0.012	123.7	122
0.013	114.0	116
0.015	98.4	99
0.017	86.0	86
0.020	70.0	71
0.030	44.4	45

The agreement of the coefficients ascertained by the author with those of the Kutter formula for the same values of n is no less striking than is the case with the exponents. It follows, for the same reason, and has no greater significance.

Those who wish to make use of the valuable data represented by this paper will do well to refer to the original publication, as the paper under discussion hardly does justice to it.

Mr. Sleight.

R. B. SLEIGHT,* JUN. AM. SOC. C. E. (by letter).—The writer has had occasion to investigate the flow of water in concrete-lined channels from the standpoint of the use of the exponential formula for velocity. The data on which this study was based are those in *Bulletin No. 194*, U. S. Department of Agriculture, by Frederick C. Scobey, Assoc. M. Am. Soc. C. E.; in *Bulletin No. 126*, U. S. Department of Agriculture, by Samuel Fortier, M. Am. Soc. C. E.; and in *Buletin No. 194*, Colorado Agricultural College Experiment Station, by Mr. V. M. Cone. An outline of the investigation and its results is offered in discussion of the paper by Mr. Ellis which has for its basic data *Bulletin No. 194* of the U. S. Department of Agriculture.

The writer has taken up only concrete-lined channels, and the expression for flow was assumed as

$$V = C_1 R^m S^n \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

The exponents for R and S were not assumed arbitrarily in either case, but an attempt was made to work out both of them, and it is believed that this was successful for the exponent of S as well as for that of R. The work was done by graphical methods in all cases where possible.

For analysis, Equation (1) was divided into two parts:

$$V = B S^n, \text{ or } S = A V^{\frac{1}{n}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (a)$$

which contains the term, S, and

$$A = k R^p \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (b)$$

to take care of the term, R.

* Denver, Colo.

Mr. Sleight.

A recombination then results in

$$S = k\, R^p\, V^{\frac{1}{n}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

with the first step in its investigation a study of $\frac{1}{n}$.

All the available data were plotted logarithmically. The experiments were divided into groups so that any one set or group would have but small variation in the value of R, for example, those with R less than 1.0 in one group, those with R from 1.0 to 1.5 in another group, etc. This is illustrated by Fig. 8. In this first analysis the values of S were plotted against values of V to get a numerical value for $\frac{1}{n}$, in the equation,

$$S = A\, V^{\frac{1}{n}}.$$

As the values of R cover a small range for any one group, other than the variables taken care of in this equation, the factor of friction (or whatever we may choose to call it) is the only variable left. The center of gravity line of the plotted points, S and V, for each group of representative canals was found, and $\frac{1}{n}$ was determined for that line. The following assumptions were then made: That the value as found for the group would approximate closely the true value of $\frac{1}{n}$ for any point or measurement in that group; that the arithmetical mean of these several values of the exponent (one for each group) could be taken as the true value for the exponent of V. The writer found this to be 1.84, and the resulting expression is then:

$$S = A\, V^{1.84} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

This checks very closely that recommended by a number of authors for closed channels or pipes, giving the exponent of S in the first form of Equation (1) as 0.54.

Analysis for values for the term, A, was then taken up graphically. These values were then used with corresponding values of R, that is, R for the canal from which Equation (*a*) was determined, in a logarithmic plotting to ascertain the exponent of R. The data used necessarily had to be such as consisted of two or more measurements at different stages of the same ditch. The correct formula would show the same frictional coefficient for the ditch lining, whether the ditch were flowing full or with only a small percentage of its capacity. The writer has found available only eight such experiments. These, with their numbers, references, etc., are given in Table 8. Fig. 9 shows the logarithmic diagram. The slopes of the lines vary somewhat, show-

Mr. Sleight.

TABLE 8.—CHANNELS INVESTIGATED FOR

Reference.*	Curve No.	Canal and authority.	Experiment No. in *Bulletin No. 194.*	Discharge, in second-feet.	Slope, *S.*	Mean velocity, *V.*
194.....	1	North Canal,	46	98.3	0.000639	3.02
		Bend, Ore.,	48	74.9	0.000729	2.87
		F. C. Scobey.	50	25.7	0.001021	2.04
194.....	2	North Canal,	45	98.3	0.000525	2.94
		Bend, Ore.,	47	74.9	0.000692	2.85
		F. C. Scobey.	49	25.7	0.00095	2.10
194.....	3	South Canal,	33	89.8	0.0718	15.52
		Uncompahgre,	34	59.7	0.0723	11.32
		V. M. Cone.	32	111.3	0.0015	4.71
Colo.....	4	Long Pond Chute,		35.8	0.02879	12.87
		Fort Collins,		78.3	0.02968	18.26
		V. M. Cone.		100.4	0.02943	17.89
				104.5	0.02990	20.21
				122.9	0.0?971	19.70
126.....	5	Ridenbaugh,†	15	50.0	0.000825	2.45
		Boise,		103	0.00015	2.32
		U. S. R. S.		230	0.0002	3.35
				382	0.00024	3.99
				103	0.0003	2.45
				382	0.000168	4.08
				376	0.0003	4.32
				318	0.000315	4.15
126.....	6	King Hill,		92.5	0.000187	2.06
		Idaho,		54.5	0.000452	2.54
		Don Bark.		54.8	0.000448	2.55
			30	54.6	0.00045	2.55
126.....	7	Boise, Main	2	470	0.000334	3.81
		Sec. No. 3,		470	0.000246	3.48
		U. S. R. S.		238	0.0003	2.67
				238	0.000204	2.35
126.....	8	Boise, Main		316	0.0003	3.08
		Sec. No. 2,	3	1 027	0.000288	4.68
		U. S. R. S.		476	0.000362	3.57
				245	0.000288	2.64
				119	0 000288	1.84
				1 209	0.000312	4.91

* References: 194 for *Bulletin No. 194*, U. S. Dept. Agri.
126 for *Bulletin No. 126*, U. S. Dept. Agri.
Colo. for *Bulletin No. 194*, Colo. Agri. Coll. Exp. Sta.

ing quite a range in the exponent of R. Again the arithmetical mean was taken, this average giving $R^{-1.63}$, and Equation (*b*) became

$$A = k\,R^{-1.63} \quad \ldots\ldots (4)$$

Substituting this value of A in Equation (3), there results

$$S = k\,R^{-1.63}\,V^{1.84} \quad \ldots\ldots (5)$$

which reduces to

$$V = C_1\,R^{0.88}\,S^{0.54} \quad \ldots\ldots (6)$$

The low extreme of the variation of the exponent of R is 0.76, the high extreme is 1.04. The low one is greater than shown in formulas of a similar type that have been recommended. At present,

THE VALUE OF THE EXPONENT OF R.

Mr. Sleight.

Hydraulic radius, R.	A, in $A = k\,R^p$.	p in $A = k_1\,R^p$.	C_1 in $V = C_1\,R^{0.88}\,S^{0.54}$.	Mean C_1.	Maximum percentage of variation from mean C_1.
1.90	0.00 0837		89		
1.62	0.00001046		92		
0.91	0.000275	— 1.65	91	91	2.2
1.94	0.0000719		97		
1.63	0.0000912		97		
0.88	0.000242	— 1.58	101	98	3.1
0.51	0.000448		115		
0.38	0.00083		109		
1.41	0.0000866	— 1.68	114	113	3.5
0.49	0.000271		166		
0.67	0.0001426		172		
0.81	0.0001474		145		
0.76	0.0001015		171		
0.86	0.0001236	— 1.68	150	161	9.9
1.80	0.0000625		130		
2.13	0.0000319		127		
2.73	0.0000238		121		
3.29	0.0000174		126		
2.06	0.0003324		140		
3.24	0.0000174		134		
3.11	0.0000206		127		
2.90	0.0000230	— 1.89	127	130	7.6
1.71	0.0000495		134		
1.25	0.0000807		135		
1.25	0.0000801		135		
1.25	0.0000804	— 1.58	135	135	0.7
2.45	0.0000284		133		
2.65	0.0000218		131		
1 87	0.0000492		123		
2.09	0.0000423	— 1.91	121	127	4.7
2.14	0.0000378		125		
3.88	0.0000167		119		
2.64	0.0000348		109		
1.95	0.0000450		120		
1.44	0.0000940		109		
4.22	0.0000167	— 1.56	109	115	8.7

† Not all measurements on same reach; slopes taken from diagrams of Kutter's n.

however, the writer can find no justification for making an average exponent lower than 0.88. The variation from the mean is large. All the measurements seem to have been made with care. The value of 0.54 for the exponent of S, when used in ascertaining the exponent of R, gave lines of a more nearly constant slope than could be obtained when any other value was used. Therefore, in adopting the values, $R^{0.88}\,S^{0.54}$, the writer believes them to fit the conditions investigated —and conditions quite generally for concrete-lined canals—more closely than other exponential expressions; but he does not claim that the exponential expression will represent exactly the flow in open channels.

Mr. Sleight.

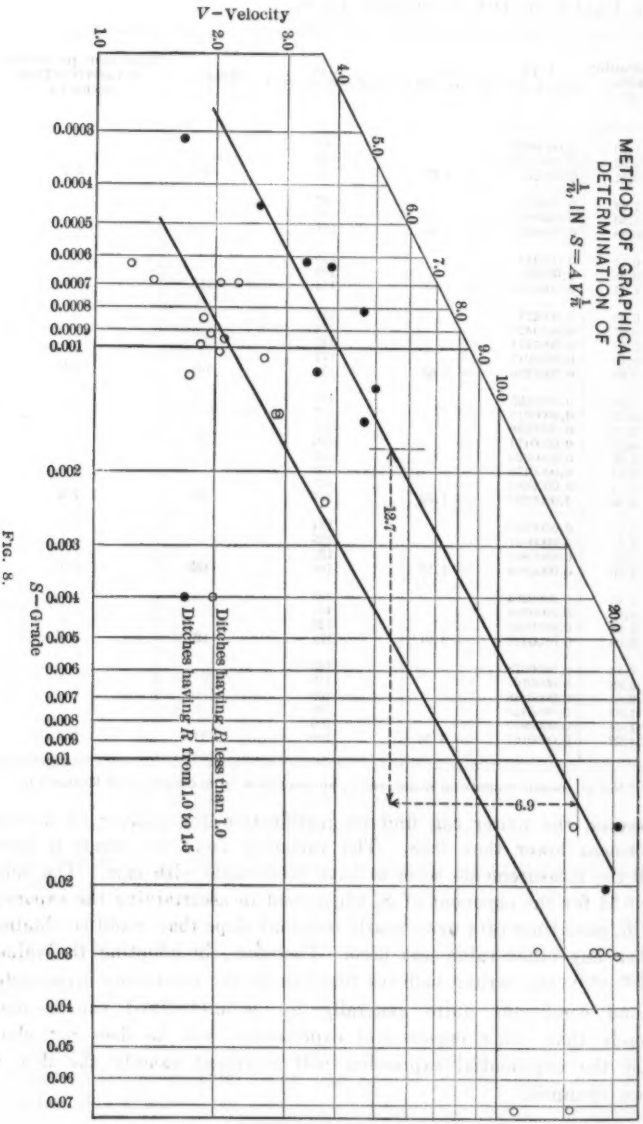


FIG. 8.

Mr. Sleight.

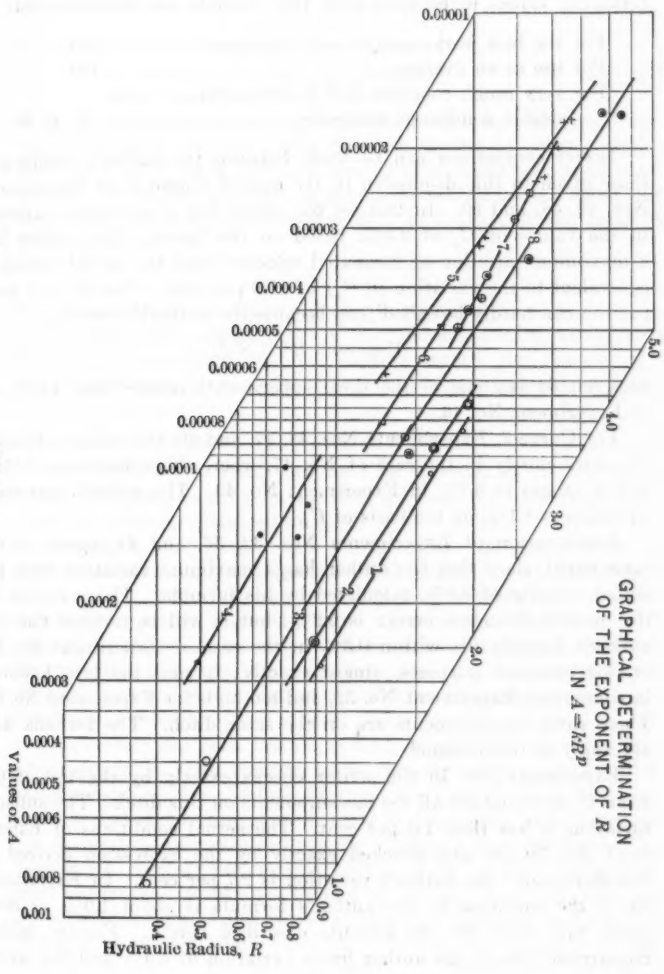


FIG. 9.

Mr. Sleight.

Equation (6) gives a comparatively constant C_1 for ditches of apparently the same character. Throughout the range of data, the following values to be used with this formula are recommended:

For the best workmanship and alignment	165
For use as an average	130
For very rough concrete, bad workmanship, or undesirable conditions generally	70 to 90

Direct comparison can be made between the author's results and those given in this discussion in the case of Curve 1, or Experiments Nos. 46, 48, and 50. In this set the writer has a maximum variation in the values of C_1 of 3.4%, based on the lowest. The author has a maximum variation of computed velocity from the actual (which is equivalent to the variation in C_1) of 17.6 per cent. Possibly this comparison can hardly be called just, because the author's formula,

$$V = 80.3\, R^{0.69}\, S^{0.5},$$

does not fit any one of the three experiments nearer than 4.64%, as in Experiment No. 46.

For Curve 2, Experiments Nos. 45, 47, and 49, the author's formula fits more closely in the case of No. 47, where the variation is 1.05%, and it ranges to 6.2% in Experiment No. 49. The writer's maximum variation is 4.1%, on the basis of C_1.

Examination of Experiments Nos. 32, 33, and 34, again on the same canal, show that the author has a maximum variation from the actual velocity of 4.5%, calculated by his formula. The variation by the method discussed herein is 5.2%; but it will be noticed that the author's formula fits within 0.3% in the case of Experiment No. 33, or, for practical purposes, almost exactly; it runs too low, however, in computing Experiment No. 32, and too high for Experiment No. 34. These three measurements are on the same ditch. The formula does not apply to this channel.

Experiment No. 15 the writer checks exactly by the use of the mean C_1 as found for all the measurements on this ditch. The author's variation is less than 1.0 per cent. The actual conditions of Experiment No. 30 are also checked exactly by the expression derived in this discussion; the author's variation is 2.7 per cent. In Experiment No. 2 the variation by the author's formula is about 1.0%, as compared with 4.5% by the formula described herein. Finally, taking Experiment No. 3, the author has a variation of 5.4% and the writer 3.4 per cent.

The last four comparisons cannot be given quite as much weight as those where more than one measurement on the same canal was

Mr. Sleight.

available. On the whole, the data for comparison are not as complete as desired; still, it seems evident that

$$V = C_1 R^{0.88} S^{0.54}$$

is nearer the correct expression than

$$V = C R^{0.69} S^{0.5}.$$

It is unfortunate that more experiments are not available on the same reach of canal, with different depths of water. Although *Bulletin No. 194* by Mr. Scobey, and such investigations as that of the author, give all the information needed on which to base canal design, they do not present the necessary data to prove that the exponential formula of the type

$$V = C_1 R^m S^n$$

will apply exactly to the flow of water in open channels.

The writer at present believes that the exponential or logarithmic expression, unmodified, will not express exactly the velocity of flow in open channels when constant exponents are attempted for all types of such channels.

Mr. Hoyt.

W. G. HOYT,* ASSOC. M. AM. SOC. C. E. (by letter).—Though this discussion deals primarily with the flow of water in natural channels, it has also some bearing on the flow of water in irrigation channels, especially those constructed of earth.

During a considerable part of the year, the channels of many streams contain a plant growth—both natural and artificial—that tends to retard the velocity of the water, increase the cross-section necessary to carry a given volume of water, and cause some local changes in slope. Apparently, such growth has sufficient effect on the regimen of the stream to change the value of n from 0.016 for earth channels in excellent condition to 0.030 for channels subject to heavy "growths of moss or other aquatic plants."

The writer has been unable to find, in any treatise on hydraulics or irrigation, any information as to the character or duration of the plant growth, or its absolute effect on the flow of the water.

Fortunately for the purpose of this discussion, but unfortunately for the purpose of collecting records of stream flow, the United States Geological Survey has maintained gauging stations at points on certain streams in Minnesota and Wisconsin where aquatic plants grow in considerable quantities during the summer, and the results obtained at these stations show the effect of the plant growth on the flow of the rivers. The gauging station on Elk River, near Big Lake, Minn., affords a typical example. Elk River, which drains an area lying chiefly in Benton and Sherbourne Counties, rises in T. 38 N.,

* Madison, Wis.

Mr. Hoyt.

R. 29 W., at approximately 1 150 ft. above sea level, flows south and east, and enters the Mississippi 20 miles up stream from Minneapolis, at an elevation of 858 ft. above sea level. At the gauging station, about 8 miles above the mouth, the drainage area is 615 sq. miles. Records collected from 1911 to 1915 show a mean flow of 200 sec-ft., a maximum of about 5 000 sec-ft., and a minimum of about 50 sec-ft. The flow during the summer varies from 100 to 300 sec-ft.

The effect of aquatic plants appears about June 1st, increases gradually through July and August, and disappears about November 1st. Between the time when the ice leaves the river and June 1st, and between November 1st and the formation of ice, a sufficient number of current-meter measurements have been made to determine the re-

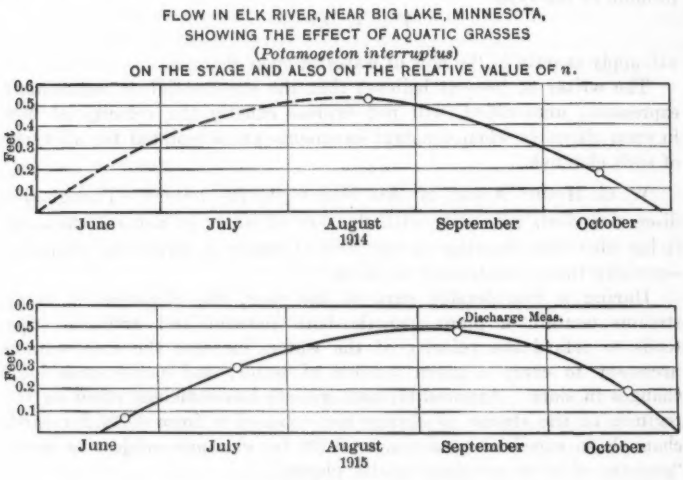


FIG. 10.

lation of flow to stage in the unobstructed channel. If the only changes in the condition of the channel are those caused by aquatic plants, it is possible to measure the effect of such growth on the stage of the river when the flow is uniform. The effect is shown graphically by Fig. 10, which has been prepared by assuming uniform conditions of flow during 1914 and 1915. The curve for 1915 is more complete than that for 1914. It is assumed that there was no back-water due to plant growth on June 1st, 1914, and November 1st. The back-water shown by the discharge measurement on August 20th was +0.53, and that of October 14th was +0.17. In 1915 the maximum effect of back-water was approximately +0.47, as shown by the discharge measurement on September 9th, and back-water from plant growth

affected the stage from June 15th to October 31st. As data relating to slope and cross-section are not available, it is not possible to show the absolute effect of this plant growth on *n*, but it is believed that the curve shows a relative effect on the value of that factor. Mr. Hoyt.

The aquatic plant most common in the streams of Minnesota and Wisconsin is *Potamogeton*, of which there are apparently sixty-five well-known species. That which grows in the Elk River is probably *Potamogeton interruptus*. The popular name is pondweed. According to MacMillan,* and Britton and Brown,† most pondweeds are submerged plants growing in ponds, lakes, and slow streams in many of the northern States and Canada. About twenty species are known in Minnesota, and some forty in Wisconsin. They are rooted in the soil under the water, rather than attached to rock or growing loose in the water. Their stems are branching, and when taken out of the water are limp, owing to poor development of woody tissues. The flowers are commonly collected on spikes, which in the spring are barely thrust above the water in order that the wind may carry the pollen from the stamens to the stigmas. Very closely related to the pondweeds are the Naiads, which may be distinguished by the solitary pistil which forms the fruit.

The growth of aquatic plants depends largely on the temperature, depth, velocity, and chemical contents of the water, the length of growing period, facility for rooting, and other factors that influence plant life. *Potamogeton* is a perennial growth, the plant dying or withering each winter, and the root remaining alive. The more luxuriant growth during 1914 was probably due to higher temperatures in that year, as the mean monthly temperature was higher than in 1915 by 4.3° in June, 8.1° in July, 6.2° in August, 1.8° in September, and 4.0° in October.

Although the aquatic growths are often spoken of as "grasses" or "mosses", few of the plants growing in streams belong to the grass family, and the "moss" said to grow in irrigation channels is undoubtedly a species of algæ.

It is not the object of this discussion to criticize Mr. Ellis' paper, but rather to submit evidence showing the effect of aquatic growth on river regimen—a subject which apparently has received little attention by engineering writers.

WILLIAM S. ALDRICH,‡ ESQ. (by letter).—This valuable paper illustrates once more the constant tendency of scientific thought in its treatment of natural phenomena. It is from the complex to the simple. After a half century of the Ganguillet-Kutter adaptation of the Chezy formula, itself then nearly a century old, we have had, in the last score of years, to reckon with Bazin's attempt to revert Mr. Aldrich.

* "Minnesota Plant Life."

† "Flora in Northern States and Canada."

‡ Fort Collins, Colo.

Mr. Aldrich.

to the simplicity of the Chezy formula. Encouraged in this logical process, the author of this paper boldly attempts the final reduction. It is, however, all the more worthy of an audience, not necessarily because it is the latest, but in that it is based on the unimpeachable data of the Government tests, conducted by the Department of Agriculture. Beyond this we may go, but scarcely at private expense, or college laboratory charge.

Examining the summary of results, Table 2, we find a tendency of the product, nC, to assume a nearly constant value; that is, this product ranges slightly above and below the value inherent in the type of canal which is usually accepted as representative of the given class of construction. This is shown in Table 9.

Plotting the values for each series, throughout the whole range, we find that the author has selected the value of C which corresponds very nearly to the general average values and the usual grade of construction, as shown in Table 10.

TABLE 9.

Type.		n.	C.	Product, nC.
Concrete	II	0.013	113.5	1.4755
	III	0.014	107.5	1.505
	IV	0.015	99.0	1.485
	VII	0.018	80.0	1.440
Wood	II	0.013	114.0	1.482
	III	0.014	106.0	1.484
	IV	0.015	99.5	1.4925
	V	0.016	91.0	1.456
Metal	I	0.011	137.0	1.507
	II	0.015	93.5	1.4025
Earth	I	0.016	90.5	1.448
	II	0.020	70.3	1.406
	III	0.0225	61.2	1.377
	IV	0.025	52.8	1.320

Referring to Fig. 3, we find, at several points along the curve, a helpful interpretation of relations between n and C. For example, the following marked points may be noted:

n.	C.	Product, nC.
0.014	105.8	1.4812
0.018	80.3	1.4454
0.0225	60.2	1.3545

These relations are more or less significant; they indicate the very close approximation of the constants selected to the actual conditions prescribed by the usual types of construction. It is thus found that the better types of wooden channels are in the same class as the

poorer types of the concrete, though even the poorer types of wooden channels hold up to the concrete remarkably well. The better types of earth channels are in the identical class of the poorer types of the wood. As might be expected, the poorer type of earth channels, within the values, 0.0225 and larger, are in a class by themselves. This grading of the relative values of the constructive types is in strict accordance with the values of the product, nC, used as a criterion, and based on Table 2. The metal channels are also clearly in a class by themselves, with this product ranging from 1.40 to 1.50, Types II and I, respectively. Mr. Aldrich.

TABLE 10.

Type.	n.	C (Ellis).	Product, nC.
Concrete channels (similar to III).....	0.014	105	1.470
Wooden channels (between III and IV).	0.0148	100	1.480
Earth channels (similar to III).....	0.0225	60	1.350

Fig. 3 is a curve of the nature of an equilateral hyperbola; that is, the product, nC = a constant (approximately). It is not intended to convey the impression that this product is, or should be, constant. The Manning formula, quoted by the author, assumes that it is a constant, and equal to 1.49. We must bear in mind that the exponent of R, in the Manning formula, is 0.67, as compared with an exponent of 0.69 in the Ellis formula.

M. J. ORBECK,* JUN. AM. SOC. C. E., and MESSRS. C. N. WARD,* and W. O'B. HENDERSON* (by letter).—In his article, "Flow of Water in Irrigation Channels",† Mr. Fred. S. Scobey has compiled a table giving the results of measurements and conditions of flow in channels of great variety, and in it the Kutter coefficient of roughness and the Chezy coefficient are included. The engineer using either of these formulas may choose a value of n or C corresponding to the channel which most nearly resembles that which he is considering. Messrs. Orbeck, Ward, and Henderson.

Owing to the fact that Manning's and Bazin's formulas are not used extensively in America, the retardation factors for these formulas were not included in Mr. Scobey's table. Either of these formulas is more convenient to use than Kutter's, which consideration led the writers to compute Manning's n and Bazin's γ for these experiments.‡ The correspondence between Kutter's n and Manning's n is quite startling, and it is probable that Manning's formula with Kutter's n, will give as accurate results as can be obtained from Kutter's formula.

* Ann Arbor, Mich.

† *Bulletin No. 194*, U. S. Department of Agriculture.

‡ These values are given in a table which is not reproduced here, but is filed in the Library of the Society.

Messrs. Orbeck, Ward, and Henderson.

Mr. Scobey's entire table has been reproduced with the exception of the columns headed "Experimenter", "Shape of Channel", "Depth Determination", "Discharge Determination", "Velocity Curve", "Temperature of Air", "Temperature of Water", and "Wind Conditions". Where such information is wanted, the reader is referred to the original publication. Columns containing Manning's n and C and Bazin's γ have been added.

All the equations considered may be written in the form:

$$V = C\sqrt{RS} \quad \ldots\ldots(1)$$

where C is given directly, as in the Chezy formula,

or by Kutter,
$$C = \frac{41.66 + \frac{0.00281}{S} + \frac{1.811}{n}}{1 + \left(41.66 + \frac{0.00281}{S}\right)\frac{n}{\sqrt{R}}} \quad \ldots\ldots(2)$$

or by Manning,
$$C = \frac{1.49}{n} R^{0.17} = C' R^{0.17} \quad \ldots\ldots(3)$$

or by Bazin,
$$C = \frac{157.6}{1 + \frac{\gamma}{\sqrt{R}}} \quad \ldots\ldots(4)$$

From Equations (3) and (4) may be derived the following:

$$n = \frac{1.49}{C} R^{0.17} \quad \ldots\ldots(5)$$

$$C' = \frac{1.49}{n} = \frac{C}{R^{0.17}} \quad \ldots\ldots(6)$$

$$\gamma = \left(\frac{157.6}{C} - 1\right)\sqrt{R} \quad \ldots\ldots(7)$$

From Equations (5), (6), and (7), Manning's n and C, and Bazin's γ, given in the table, were computed, the value of C being that given by Mr. Scobey.

These computations were made in connection with a graduate course at the University of Michigan, under the general supervision of H. W. King, M. Am. Soc. C. E., Professor of Hydraulic Engineering.

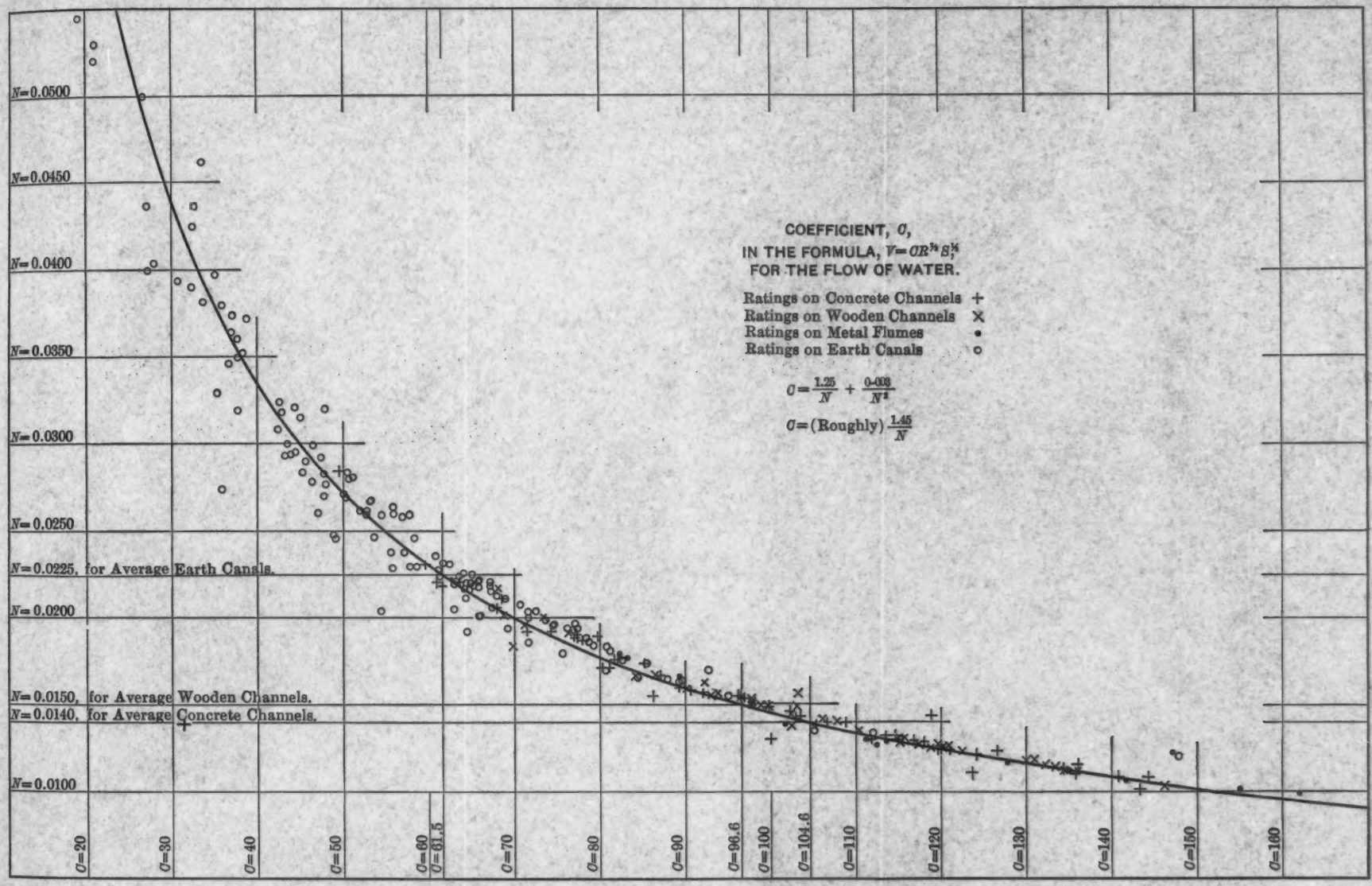
Mr. Ellis.

GEORGE HENRY ELLIS,* ASSOC. M. AM. SOC. C. E. (by letter).—In any discussion of data of this sort, some classification is almost necessary. It seemed logical to make this classification according to the roughness of the channel, and also to base any new formula on experience obtained in the use of an old one. The writer, therefore, arranged the channels in accordance with their computed values of n, which is the method to which Mr. Hazen objects.

The writer agrees with Mr. Harding and Mr. Aldrich that it is desirable to have an expression of the relation between C and n,

* Fort Shaw, Mont.

PLATE LXI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXX, No. 1373.
ELLIS ON
FLOW OF WATER
IN IRRIGATION CHANNELS.



at least until we become accustomed to thinking in terms of C; and is indebted to both these gentlemen for the formulas submitted by them. Mr. Ellis.

Mr. Harding mentions the Williams formula, $V = CR^{0.67} S^{0.54}$, as though it were well known. The writer did not find it, either in Merriman (Ninth Edition) or in Parker's "Control of Water", both standard works on hydraulics; and had not seen it before. The object of this paper was to find a simpler formula for the flow of water, not necessarily the one deduced in the paper, but one which could be accepted by the Profession. Perhaps the Williams formula is the one. It has the different exponent for S which it was hoped would make a better curve of the points in Fig. 4. The exponent of R, 0.67, is well within the range of these points, and might fit them still better had they been obtained by $S^{0.54}$ instead of by $S^{\frac{1}{2}}$. Mr. Harding has deduced a coefficient, also, for it, $C = \frac{1.70}{n - 0.002}$, so that it can be used directly with n.

It was thought that where extreme accuracy was not required, the Williams formula might be reduced to $V = CR^{\frac{2}{3}} S^{\frac{1}{2}}$, which could be solved on almost any slide-rule, and Tables 11 and 12 have been arranged in order to compare the effects of the various exponents. The difference between $S^{0.54}$ and $S^{\frac{1}{2}}$ is surprising. Perhaps this latter expression, which is essentially the Manning formula, if used with a different coefficient, would give results sufficiently close for all practical purposes.

TABLE 11.—POWERS OF RADII.

R	$R^{0.69}$	$R^{0.67}$	$R^{\frac{2}{3}}$
0.2	0.3294	0.3402	0.3420
0.3	0.4357	0.4464	0.4481
0.4	0.5314	0.5412	0.5429
0.6	0.7030	0.7102	0.7114
0.8	0.8573	0.8611	0.8618
1.0	1.0000	1.0000	1.0000
1.5	1.3228	1.3122	1.3104
2.0	1.6133	1.5912	1.5874
3.0	2.1341	2.0878	2.0801
4.0	2.6027	2.5316	2.5198

The writer computed the values of C in Manning's formula, $V = C R^{\frac{2}{3}} S^{\frac{1}{2}}$, for each experiment, before he knew that it had already been done by Mr. Orbeck and his colleagues. The results are plotted on Plate LXI, on which the curve shows the relation between Kutter's n, as given by Mr. Scobey, and Manning's C, of $C = \frac{1.45}{n}$, roughly, and

Mr. Ellis.

this resembles the expression found by Mr. Aldrich for the writer's curve, Fig. 3. The break in the curve at about $n = 0.0225$, mentioned by Mr. Aldrich, is also quite noticeable on Plate LXI. The plotted curve is the expression, $C = \frac{1.25}{n} + \frac{0.003}{n^2}$, and is worked out in Table 13.

TABLE 12.—POWERS OF SLOPES.

S	$S^{0.54}$	$S^{\frac{1}{2}}$	S	$S^{0.54}$	$S^{\frac{1}{2}}$
0.00010	0.0069	0.0100	0.003	0.0434	0.0548
0.00013	0.0080	0.0114	0.004	0.0507	0.0632
0.00016	0.0089	0.0126	0.005	0.0572	0.0707
0.00020	0.0100	0.0141	0.006	0.0631	0.0775
0.00025	0.0113	0.0158	0.008	0.0737	0.0894
0.0003	0.0125	0.0173	0.010	0.0832	0.1000
0.0004	0.0146	0.0200	0.013	0.0958	0.1140
0.0005	0.0165	0.0224	0.016	0.1072	0.1265
0.0006	0.0182	0.0245	0.020	0.1209	0.1414
0.0008	0.0213	0.0283	0.025	0.1364	0.1581
0.0010	0.0240	0.0316	0.03	0.1505	0.1732
0.0013	0.0276	0.0360	0.04	0.1758	0.2000
0.0016	0.0309	0.0400	0.05	0.1983	0.2236
0.0020	0.0349	0.0447	0.06	0.2189	0.2450
0.0025	0.0393	0.0500	0.10	0.2884	0.3162

TABLE 13.

n	C	n	C	n	C	n	C
0.0090	170.5	0.0140	104.6	0.0225	61.5	0.0400	33.2
0.0100	155.0	0.0150	96.6	0.0250	54.8	0.0450	29.3
0.0110	138.4	0.0160	89.9	0.0275	49.5	0.0500	26.2
0.0120	125.0	0.0180	78.8	0.0300	45.0	0.0550	23.7
0.0130	114.0	0.0200	70.0	0.0350	38.2		

The writer confesses that he has been converted to the simple Manning type of formula, as its use does not require tables, diagrams, or even a special slide-rule. Computations for Plate LXI were made on an ordinary slide-rule having scales of squares and cubes. If this expression can be generally accepted and used, the term, n, will probably be gradually dropped, and in designing a concrete-lined section, for example, the velocity may be computed at $105\ R^{\frac{2}{3}}\ S^{\frac{1}{2}}$, which is nearly as simple as it can be made.

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Paper No. 1374

METHOD OF DESIGNING A RECTANGULAR REINFORCED CONCRETE FLAT SLAB, EACH SIDE OF WHICH RESTS ON EITHER RIGID OR YIELDING SUPPORTS*

By A. C. Janni, M. Am. Soc. C. E.

With Discussion by Messrs. Edward Godfrey, E. P. Goodrich, Ernst F. Jonson, Charles M. Montgomery, C. A. P. Turner, Henry T. Eddy, L. J. Mensch, Charles F. Marsh, Franklin R. McMillan, and A. C. Janni.

Synopsis.

This paper deals with the incorrect method generally followed in calculating the moments of a continuous reinforced concrete flat slab resting on a system of girders.

That the elastic displacement of the supports should not be ignored, is demonstrated by theory and by a practical application.

All building codes, as far as the writer knows, overlook this important fact, and something should be done to correct the error.

There is no possibility of giving fixed formulas for designing this kind of construction; only the general lines of the method to be followed can be indicated.

Let *EFGH*, Fig. 1, be a flat slab floor, the reinforcement being parallel to *EF* and *FG*, supported by four perimetral walls, *EF*, *FG*, *GH*, *HE*; four columns, *K*, *L*, *N*, *M*; and girders as shown.

* Presented at the meeting of April 5th, 1916.

Taking into consideration the portion, *KLNM*, of that slab, suppose, for the present, that the four girders, *KL*, *LN*, *NM*, *MK*, which support it, are absolutely rigid.

Suppose now that this slab is divided into two systems of beams: System *a* parallel to the side, *KL*, and System *b* parallel to the side *LN*, as shown in Fig. 1.

If this slab carries a uniform load, *w*, per square foot, it is clear that the deflection of the beams of System *a* will not be the same for all of them, but will decrease from the center of Span *l* toward the sides, *KL* and *NM*.

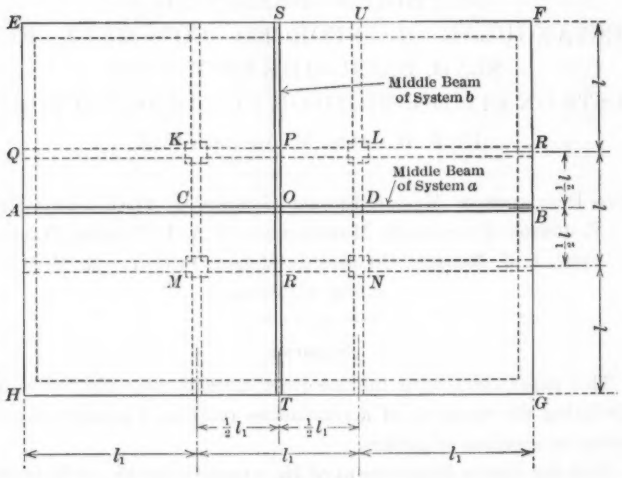

FIG. 1.

A similar condition will exist among the beams of System *b*.

Therefore, although these beams, *a* and *b*, are carrying the same uniform load, their deflections are different.

The solution of this problem, therefore, consists in finding a certain coefficient, α, by which the uniform load, *w*, should be corrected, in order to comply with the behavior of the slab.

Now, take into consideration the two beams, *CD* and *PR*. Each, being at the middle line of the slab, is the most deflected of its system; and, for the above consideration, if *w* is the uniform load assumed, αw will be the load at the center of the span, and *w* the load at the

ends. Then $(1 - \alpha)\ w$ will be the load at the center of the span of the other beam, and w that at its ends.

It will be assumed that the law of variation of the load is a parabolic one.

Let PO, Fig. 2, be the half span, PO, of PR, Fig. 1; this half beam, on account of the symmetry of the beam, PR, may be regarded as a cantilever fixed at P and under the action of a hyperstatic moment, M_o.

If F is an ideal force applied to the cantilever at O, the deformation, D, is given by:

$$D = \frac{d\,L}{d\,F} \qquad \ldots\ldots(1)$$

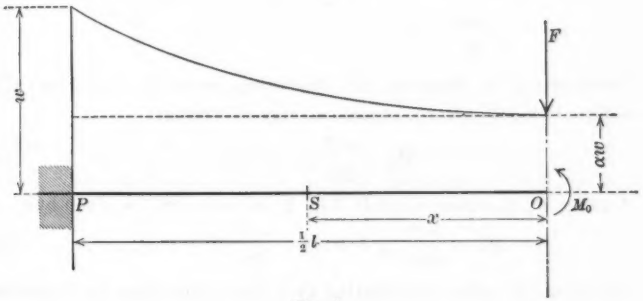

FIG. 2.

Disregarding the deformation caused by shear, and calling $\frac{1}{m}$ the coefficient of Poisson, it is known that the expression of the work, L, is given by:

$$L = \int_0^{\frac{l}{2}} \frac{m^2 - 1}{m^2} \times \frac{M^2\,dx}{2\,E\,I} \qquad \ldots\ldots(2)$$

Equation (1), therefore, may be written:

$$D = \frac{m^2 - 1}{m^2\,E\,I} \int_0^{\frac{l}{2}} M\,\frac{d\,M}{d\,F}\,dx \qquad \ldots\ldots(3)$$

For any generic section, S, Fig. 2, the value of M and $\frac{d\,M}{d\,F}$ are given by:

$$\left.\begin{aligned} M &= M_o - \alpha\,w\,\frac{x^2}{2} - \frac{w}{3}(1 - \alpha)\,\frac{x^4}{l^2} - Fx \\ \frac{d\,M}{d\,F} &= -\,x \end{aligned}\right\} \qquad \ldots\ldots(4)$$

In the foregoing expression of M there is the unknown quantity, M_O, which may be determined by the condition that the derivative of the work, L, of deformation, with respect to the hyperstatic moment, M_O, must be zero, that is to say:

$$\frac{d\,L}{d\,M_O} = 0 \qquad (5)$$

Taking the value of L from Equation (2), then Equation (5) will become:

$$\int_0^{\frac{l}{2}} M \frac{d\,M}{d\,M_O}\,dx = 0 \qquad (6)$$

where:

$$\left.\begin{aligned} M &= M_O - \alpha\, w\, \frac{x^2}{2} - \frac{w}{3}(1-\alpha)\,\frac{x^4}{l^2} \\ \frac{d\,M}{d\,M_O} &= 1 \end{aligned}\right\} \qquad (7)$$

Substituting in Equation (6) the values given by Equations (7), then Equation (6), after integration, may be written:

$$M_O = \frac{w\,l^2}{240}(1 + 9\,\alpha) \qquad (8)$$

Calling M_P the moment at P, Fig. 2, its value will be given by:

$$M_P = \frac{w\,l^2}{240}(1 + 9\,\alpha) - \frac{w\,l^2}{48}(1 + 5\,\alpha) \qquad (9)$$

Equation (3), after substituting in it the values given by Equations (4) and (8), performing the integration, and putting $F = 0$, will become:

$$E\,I\,D = \frac{m^2 - 1}{m^2} \times \frac{w\,l^4}{2\,880}(1 + 6.5\,\alpha) \qquad (10)$$

A similar expression can be obtained for the beam, $C\,D$, having the span, l_1; that is to say:

$$E\,I\,D = \frac{m^2 - 1}{m^2} \times \frac{w\,l_1^4}{2\,880}[1 + 6.5\,(1 - \alpha)] \qquad (11)$$

But the two beams, $P\,R$ and $C\,D$, must have the same deflection at O, therefore Equations (10) and (11) will furnish the other equation:

$$(1 + 6.5\,\alpha)\,l^4 = [1 + 6.5\,(1 - \alpha)]\,l_1^4$$

or

$$\alpha = \frac{7.5 - \left(\frac{l}{l_1}\right)^4}{6.5\left[1 + \left(\frac{l}{l_1}\right)^4\right]} \qquad (12)$$

Equation (12) becomes $\alpha = 1$, if $\frac{l_1}{l}$ is about $\frac{5}{3}$, that is to say, when the ratio, $\frac{L\ N}{K\ L}$, is larger than $\frac{5}{3}$, there is no advantage in the use of two-way reinforcement.

If $l = l_1$, then $\alpha = \frac{1}{2}$; that is to say, the advantage of two-way reinforcement is maximum when the slab is square.

As one cannot depend always on the effectiveness of the fixed end, P, the value of M_O, given by Equation (8), should be taken for the practical application of that equation, by assuming that the moment at P is only two-thirds of that given by Equation (9); therefore, this moment should be:

$$M_O = \frac{w\ l^2}{240}(1 + 9\ \alpha) + \frac{w\ l^2}{180}(1 + 4\ \alpha) \ldots\ldots\ldots\ldots (13)$$

or

$$M_O = \frac{w\ l^2}{720}(7 + 43\ \alpha) \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

The values of M_P and M_O, given by Equations (9) and (14), are those to be substituted in the general equation of stability given by theory:

$$f\ \frac{I}{h} = \frac{m^2 - 1}{m^2}\ M \ldots\ldots\ldots\ldots\ldots\ldots (15)$$

where, f = maximum allowable stress,

I = moment of inertia,

h = distance between the neutral axis and the farther fiber,

m = coefficient of Poisson,

M = external moment.

Assuming $m = 4$, Equation (15) will become:

$$f\ \frac{I}{h} = \frac{15}{16}\ M \ldots\ldots\ldots\ldots\ldots\ldots (16)$$

Substituting in Equation (16), and giving to M the values found by Equations (9) and (14), the following equations are derived:

$$\left.\begin{aligned} f\ \frac{I}{h} &= \frac{w\ l^2}{64}(1 + 4\ \alpha) \ldots\ldots\ldots\ldots \text{at } P \\ f\ \frac{I}{h} &= \frac{w\ l^2}{768}(7 + 43\ \alpha) \ldots\ldots\ldots\ldots \text{at } O \end{aligned}\right\} \ldots\ldots (17)$$

If $l = l_1$, then, from Equation (12), $\alpha = \frac{1}{2}$, and Equations (17) will become:

$$\left.\begin{aligned} f\frac{I}{h} &= \frac{1}{21} w l^2 \text{..... at } P \\ f\frac{I}{h} &= \frac{1}{27} w l^2 \text{..... at } O \end{aligned}\right\} \text{.....(18)}$$

Having obtained, in each case, the value of α from Equation (12), by using Equations (17), the moments at the center of the span and on the supports may be found, and the calculation of the sections of the slab may be carried out by the usual methods.

Thus far, the supports of the slab, *KLNM*, have been supposed to be rigid; and, as long as this rigidity is to be expected, Equations (12) and (17) may be used legitimately; but the problem, as stated at the beginning of the paper, goes a little farther.

In fact, the four girders, *KL*, *LN*, *NM*, *MK*, supporting the panel, *KLNM*, are not rigid; they deflect with the loading of the panel, or panels, and this deflection affects, in no negligible way, the external moments as given by Equations (17), as will be shown later.

Suppose the beam, *CD*, Fig. 1, to be extended on each end to *A* and *B*, the ends, *A* and *B*, of this ideal beam, *ACDB*, being supported by the walls, *EH* and *FG*.

If D_1 is the deflection of the girder, *KM*, at the middle of its span, this deflection—considered as a deflection at *C* of the beam, *AB*—corresponds, according to theory, to a moment at Section *C* of this beam, given by:

$$M''_C = \frac{6}{5} E I_1 \frac{D_1}{l_1^2} \text{.....(19)}$$

Hence, at Section *C* of the beam, *AB*, there will be a total moment given by:

$$M_C = M'_C + M''_C \text{.....(20)}$$

The vertical reaction at *A* will be:

$$V_A = \frac{1}{2} w l_1 + \frac{M_C}{l_1} \text{.....(21)}$$

and, therefore, the maximum moment of the span, *AC*, will be:

$$M_{AC} = \frac{V_A^2}{2w} \text{.....(22)}$$

and the value of the moment at the middle of the span, CD, is found to be:

$$M_O = \left(V_A - \frac{3}{8} w l_1\right) l_1 \ldots\ldots\ldots\ldots\ldots\ldots (23)$$

The reactions and moments for the beam, ST, are to be computed in a similar way.

The following is a practical application of the foregoing method:

Let $EFGH$, Fig. 1, be a floor system, as supposed herein, and, for a practical application of the formulas shown, let $l = 18$ ft. and $l_1 = 24$ ft.

The dead load plus live load will be assumed to be 300 lb. per sq. ft.

The moments acting on the ideal beam, PR, of the panel, $KLNM$, will be found first on the assumption that its supports at P and R are unyielding.

Equation (12) gives:

$$\alpha = 0.83.$$

Equations (17), consequently, give:

$$M_P = 6\ 561 \text{ ft-lb.}$$
$$M_O = 5\ 402.8 \text{ "}$$

In the foregoing expression for α, it should be noted, the ratio $\frac{l}{l_1}$, must be taken always so that $\frac{l}{l_1} < 1$, since the values of α cannot logically vary except between $\frac{1}{2}$ and 1. Therefore, after α is obtained and the moments for the shorter span are computed, the moments for the other beam may be determined by establishing the relations:

$$\frac{M_O}{M'_O} = \frac{l^2 (7 + 43\ \alpha)}{l_1^2 (50 - 43\ \alpha)},$$
$$\frac{M_P}{M'_C} = \frac{l^2 (1 + 4\ \alpha)}{l_1^2 (5 - 4\ \alpha)}.$$

The moments at C and O for the beam, CD, therefore, will be found to be:

$$M'_C = 4\ 588.1 \text{ ft-lb.}$$
$$M'_O = 3\ 254.7 \text{ "}$$

Taking under consideration now the beam, CD, on account of the deflection of the girder, KM, the moment at C and, consequently, the moment at O, will assume other values which must be determined.

Admitting that the deflection of the girder, KM, at its middle point, C, is 0.105 in., the moment at C, M''_C, from the beam, AB, causing this deflection, is given by:

$$M''_C = \frac{6}{5} \times 2\,000\,000 \times 722.4 \times \frac{0.105}{288^2} = 183 \text{ ft-lb.}$$

Therefore, the total moment at C will be:

$$M_C = M'_C - M''_C = 4\,588.1 - 183 = 4\,405.1 \text{ ft-lb.}$$

and the vertical reaction at A will be given by:

$$V_A = \frac{1}{2} w l_1 + \frac{M_C}{l_1} = \frac{1}{2} 300 \times 24 - \frac{4\,405.1}{24} = 3\,416.5 \text{ lb.};$$

consequently, the moment (maximum) for the span, AC, and that at O for the span, CD, will be, respectively:

$$M_{AC} = \frac{3\,432^2}{2 \times 300} = 19\,454.1 \text{ ft-lb.}$$

$$M_O = 24 \left(3\,416.5 - \frac{3}{8} \times 300 \times 24\right) = 17\,196 \text{ ft-lb.}$$

In a similar way, the moments relative to the beam, ST, could be corrected.

If the girders supporting these panels are imagined as suppressed, then the construction becomes what is commonly termed "Flat Slab Construction", the advantages of which have been rather exaggerated.

Of course, in the case of flat slab construction, beams like AB and ST would not have the greatest deflections, and other considerations should be necessary in order to carry out the design of this slab in a satisfactory way.

In the foregoing example, it will be noted, the variations of the moments of the beam, AB, due to the deflection of the girders, KL and MN, have not been taken into account.

The variations, or corrections, due to this deflection, which could be determined by the "Principle of Reciprocity" of Maxwell, together with the variations due to the deflection of the girders, KL and MN, as it has been shown in the foregoing, are enough to persuade a designer that the calculation of the moments in a reinforced concrete floor system cannot be carried out with any "ready made" set of formulas, but that each and every case is a problem by itself.

DISCUSSION

Mr. Godfrey.

EDWARD GODFREY,* M. AM. SOC. C. E. (by letter).—At the conclusion of Mr. Janni's abstruse mathematical paper he states that:

"* * * the calculation of the moments in a reinforced concrete floor system cannot be carried out with any 'ready made' set of formulas, but that each and every case is a problem by itself."

In solving a problem, Mr. Janni has found it necessary to go nearly to the mathematical limit, even using the principle of least work, which, being interpreted, means the greatest work for the practical designer to understand.

Perhaps not one practical designer in a hundred can follow the author's analysis. Those who can are generally teaching mathematics or are mathematical prodigies. Every author of a technical paper should recognize the fact that practical designers, as a class, especially designers of building work, have not the time nor the inclination—many of them have not the capacity—to follow intricate mathematical processes. For them, formulas must be pre-digested; and, after all, are not all such formulas really put into use—when they do find use—by every-day designers?

There are some instances of designing work where the higher mathematics and complex formulas are necessary, such as for suspension and cantilever bridges, and, to some extent, draw-bridges; but these are somewhat rare, and are not touched by average designers. An attempt to introduce corresponding problems into ordinary building design is apt to result in the attempt reposing just where it originated—in the journal of an engineering society.

Mr. Janni might have solved this problem without the use of the principle of least work, for the benefit of those of the "old school" who were not taught on those lines. The writer, for one, has never had occasion to use it, as he has been able to solve such problems by simpler methods. Taking the author's expression for M in Equation (4), leaving out Fx, and using the familiar form,

$$\frac{d^2 y}{d x^2} = \frac{M}{E I},$$

the first integration gives the value of M_O identical with his, when equated to 0, with $x = \frac{l}{2}$; and the second integration gives the equation of the elastic line, from which the value of $E I D$ may readily be written. This value is identical with that by the author in Equation (10). The process is very much simpler, and more easily understood.

* Pittsburgh, Pa.

Mr. Godfrey.

The writer believes that he was the first to use this method of treating a slab supported on four edges, namely, by taking the two strips across the middle of the rectangle and observing that the load on the middle square unit is divided between these two; also, that each strip carries an increasing load toward its supports, which may be assumed to vary as the ordinates of a parabola. He published this method for a square slab in 1907, and it was promptly inserted in a technical handbook,* without giving proper credit. In 1908 he applied the same method to an oblong slab freely supported on the edges. This was just in time for the Joint Committee, in its 1909 report, to use what the writer deduced, namely, when the sides of the oblong are as 2 to 3, the full load is taken in the short direction, but the Committee clung to this classical error:

$$r = \frac{l^4}{l^4 + b^4}.$$

This error will be referred to later.

Mr. Janni would have done better to ignore Poisson's ratio, which is purely theoretical, and has never had any practical demonstration in any tensile test. The writer has never found a single engineer who would apply Poisson's ratio in the case of a hollow sphere under internal pressure and reduce the thickness of the shell to less than that demanded by the ordinary strain. This is the ideal case where that ratio would apply, if it applies anywhere. This ratio is intended for a material of uniform strength and elasticity in all directions. It could not possibly apply to a combination of concrete and rods, the latter being in only two directions. The tests which appear, to some, to show the effect of Poisson's ratio show merely the effect of tensile strength in the concrete.

The writer does not see any necessity for the assumption that the supporting beams deflect and the resulting complexity introduced by it, especially in view of the very rough assumption near the top of page 1693, that the end moment is two-thirds of the fixed-ended condition. The writer does not follow all the process at the bottom of page 1694; too much is left to the imagination. It appears that the $A\ C$ portion of the beam has a uniform load. The reason for this is not evident, since this is also a middle strip in a rectangular slab.

By leaving out of consideration the deflection of the supporting girders, the solution would be greatly simplified. The moment at the middle point of a critical strip will then lie somewhere between that found by the author (ignoring Poisson's ratio) for the fixed-ended condition and that found by the writer† for the free-ended condition. A far more potent factor which affects this moment is the unloaded

* "Principles of Reinforced Concrete Construction", by Edward R. Maurer and F. E. Turneaure.

† "Concrete", 1908, p. 289.

condition of surrounding panels. In ordinary design, free-ended beams have the coefficient, $\frac{1}{8}$, in the bending moment; this is reduced to $\frac{1}{10}$ and $\frac{1}{12}$ for interior panels. Why could not similar reductions be allowed for flat slabs? This would take the problem out of the province of abstruse mathematics and place it where a building code could reach it. Mr. Godfrey.

Mr. Janni's solution is based on fixed-ended beam strips. This applies, not only to the moment, but to the value of α. To assume the beam strips only partly fixed-ended alters the value of α. Though this alteration is too small to need consideration, it is perhaps greater than the effect of deflection of the beams.

Mr. Janni finds that when the ratio of the two span lengths is as 3 to 5, the strip in the short direction will take all the load, that is, $k = 1$. The writer's solution, for freely supported edges, gives this value unity when the ratio is 2 to 3. The Joint Committee Report gives a formula which has already been referred to as a "classical error". This formula is based on assumed equal deflection of uniformly loaded strips. The strips would have to be free to deflect as simple beams. Two or three minutes of thoughtful consideration ought to convince any one familiar with the principles of mechanics that the derivation of this formula is totally wrong; and yet it permeates the entire literature of reinforced concrete. It is in the Joint Committee Report, in "Standard Practice" issued by the American Concrete Institute, in practically all the building codes, and in nearly all the books by standard authors. This error was first propagated by European regulations, so that it is international. The writer pointed it out* and has repeatedly called attention to it.

This error is only one sample of how the literature of this subject was slapped together. Authors, code writers, and committees eagerly seized on anything, with no investigation whatever as to its correctness.

This is how the plain concrete column, the rodded column, the stirrup, the bunching of steel rods in the bottom of a beam—all of them absolutely dangerous—have become, and to a large extent remain, standards, to the everlasting shame of the men responsible and to the detriment of the best form of building construction ever devised.

A peculiarity about the Joint Committee Report and "Standard Practice" is that, coupled with this erroneous formula, the rule is given that when the sides of the rectangle are as 2 to 3, the full load is to be taken the short way, in spite of the formula which shows only

* *Engineering News*, June 3d, 1909.

Mr. Godfrey.

83% of it going in that direction. A little matter of an error of 20% does not seem to bother a set of men whose only answer to criticism was to repeat exactly the same indefensible errors four years later.

It takes about ten years for an error to be eradicated after it has once been espoused by the men who consider themselves authorities, no matter how clearly it is demonstrated. The writer hammered about that long on under pressure on dams before it began to be acknowledged to be the standard thing to do, namely, to design a dam to resist upward pressure. He has been hammering nearly that long on the rodded column and the stirrup, and the point has been reached where no engineer who reads English will dare to take up a public defense of either the stirrup or the rodded column.

The flat slab, supported on all edges on girders, does not find very many users. The flat slab supported on posts does find many users, and these users have set up a series of unscientific tests, which have been interpreted to suit themselves, and on which to base their standards of design. Then they have to call in Poisson's ratio, and invent such terms as bond-shear, and set up hypotheses as to new and strange properties of the combination of steel and concrete, all to hide the fact that tension in the concrete is performing the work that they would fain have engineers believe is being performed by the steel because of the "patented" way in which it is placed.

It is beginning now to be realized that there is a static limit below which the bending moment in a flat slab cannot possibly lie. The writer pointed this out,* in 1912, and has reiterated it at different times since then. Proponents of the flat slab, who can pass their hand over a bending moment and then show the audience that it is only half as great as before, and then repeat the trick on the same bending moment, no longer awe an audience of engineers. As the writer pointed out† in 1915, the concrete in a whole slab may be taking six times as much tension as the steel. These advocates of flat slabs attribute all this work to the steel because, forsooth, "tension in concrete is not to be relied upon permanently"—the very reason that all tension in the concrete should be accounted for and additional steel supplied to take it, for the sake of safety.

Tests ought to be made by the Federal Government, not by patent owners, interpreted by their hired experts. These tests ought to be on actual structures with columns only capable of sustaining one floor—not great first-story columns in a high building, or large piers sustaining a small slab, for the columns play a large part in flat slab tests; not spot loads in the middle of a wide expanse of floor; and not exterior panels supported by deep girders on walls. All these misleading features characterize the tests of record. No long row

* *Engineering News*, February 29th, 1912.

† In a paper before the American Concrete Institute in February, 1915.

of panels completely across a building has been tested. No exterior line of panels without a deep girder or a wall has been tested. Mr. Godfrey.

The writer made the recommendation to the Director of the Bureau of Standards, in the presence of a number of concrete engineers, that the Bureau put up sample structures and test them to destruction. No high-cost testing machine, no matter how perfect, can ever discover what a rodded column will do in a building, for, the more perfect the machine and the more ideal the test, the less the value of the results.

Tests where stress in the steel is measured have very little value, for the following reasons:

(*a*) The concrete is at the same time taking many times as much tension as the steel.

(*b*) The concrete may crack, and then all of this tension is in the steel.

(*c*) Measurements with a strain gauge, through the medium of two holes dug into the concrete to the steel, do not, in fact, tell the actual unit stress in the steel, but only the average unit stress in the measured length. If a crack occurs in this length, the unit stress may be very greatly increased for a very short length, but the effect in elongation of this is distributed over the full gauged length. This phase of the result of strain gauge measurements on embedded steel has received no attention.

Flat slab rules of design are being codified, and it behooves independent engineers to take up the question before patent owners mess it up, as they have done in the case of general design in reinforced concrete. Present standards, with their plain concrete columns, their rodded columns, their stirrups or short shear members, their rods bunched in the bottom of beams—every one wreck breeders—are an outrage and a disgrace. Engineers who are interested in sustaining the honor of the Profession should do all in their power to stem the tide of wrecks that has flowed in the last ten years, due to bad design—almost wholly standard design—by demanding safe design.

To this end the theory of flat slabs ought to be simplified, rather than the reverse. The simplest criterion is to consider a slab on rows of columns just as a slab would be considered on rows of beams. Commercial rules and the Chicago ruling will not meet this criterion, and no engineer has appeared to show why they should not.

E. P. Goodrich,* M. Am. Soc. C. E. (by letter).—For the last five or six years the writer has used a method of design for flat slab systems of reinforced concrete which takes account of deflections of interior and exterior girders, and even of the bending moments of columns. To be sure, it is more or less approximate, and the final result is secured through a method of continued approximations. Mr. Goodrich.

* New York City.

Mr. Goodrich.

A start is made with the same assumptions as those by Mr. Janni, that a rectangular slab may be divided into a system of beams at right angles to each other. The live and dead loads are assumed to be concentrated at the intersections of the beams of the two systems. Simultaneous equations have been developed giving the proportional loads supported on each beam of each system computed on the basis of the relative deflections of the several beams and the fact that the total load carried by the two systems must be the total load on the structure.

This method ignores the deformation caused by shear, and all lateral dilation, as measured by Poisson's ratio. Designs have been worked out which include: beams rigidly supported; beams with supports subject to settlement or deflection; beams with settled supports in which the tangent remains at a constant or a changing angle, etc., etc.

It is believed that Mr. Janni has done a service to the Profession by calling attention to the necessity of considering the deflection of supports in ordinary problems of design.

Mr. Jonson.

ERNST F. JONSON,* ASSOC. M. AM. SOC. C. E. (by letter).—The author calls attention to two facts which are not generally recognized, *viz.*, that the deflection of the girders and Poisson's ratio enter as determining factors into the calculation of concrete slabs reinforced in two directions. Unfortunately, however, he ignores several equally, or even more, important facts, so that his treatment of the problem can by no means be regarded as a solution. These ignored facts are the following:

The distance, $d\ k$, to the neutral axis of a reinforced concrete beam is not a constant if the concrete shrinks during hardening and drying, and practically all concrete does. The value of k becomes then a function of the moment (unfortunately, a very complex function), varying from 0 for $M = 0$ to the usually assumed values when the moment reaches its maximum. From this fact it follows, of course, that the moment of inertia is not only a constant, but also a function of the moment, decreasing about 50% in value from zero moment to maximum moment. The consequence of this condition, again, is that the theorem of three moments cannot be applied to reinforced concrete, nor any other method which deals in deflection components. The author's method, therefore, is fundamentally erroneous, and so are all calculations of statically indeterminate structures of reinforced concrete. All reinforced concrete construction is statically indeterminate with regard to the stresses, but, in speaking of statically indeterminate construction, the writer means to refer only to cases in which the loads and moments are also statically indeterminate.

* New York City.

Mr. Jonson.

In a reinforced concrete slab, the moment of inertia cannot be assumed to be the same in both directions, as is done by the author, because economy demands that less reinforcement be used longitudinally than transversely, and because the two sets of reinforcement cannot be placed on the same level.

The author's method implies the assumption that all moments are at a maximum when the entire floor is covered by the live load. This assumption is erroneous. The moment in the center of the slab is a maximum when the adjoining slabs carry no live load, and the moments over the girders reach their maximum when the live load covers the two panels between which the girders run, but not the adjoining ones.

As the slab is regarded as having fixed edges, its flexure must be one of reversed curvature. The law of variation of loading, therefore, is more closely approximated by a straight line from end to middle than by a parabola, as assumed by the author. The latter would be the proper approximation for a non-continuous slab merely supported along its edges.

All calculations of reinforced concrete made on the assumption that the ratio, n or E divided by E_c, is a constant are unreliable, because this ratio varies with the time the load is applied, from an initial value somewhat less than that usually assumed to a final value about twice as great as that generally used in reinforced concrete calculations.

It is a question whether enough is known about Poisson's ratio of concrete to justify its use as a constant in a calculation of stresses in reinforced concrete. Short-time tests have given values of from 0.25 to 0.2, but it is not known how much its value might be reduced by prolonged application of load. It may probably fall to 0.1, or even less.

The statement on page 1690 that "these beams, a and b, are carrying the same uniform load" is erroneous. The author probably intended to say, "because the beams, a and b, deflect the same, although their lengths are different, they cannot carry the same load."

The author's statement, that when the length is great enough to make $\alpha = 1$ there is no advantage in two-way reinforcement, is misleading, because it implies that there is an advantage in such reinforcement when the length is less. According to the author's method of figuring, even in the most favorable case, *viz.*, the one in which the slab is square, it requires about one-third more reinforcement than a slab reinforced in one direction only. This excess might indeed be reduced by increasing the spacing of the rods toward the edges, but, within the limits fixed by the building law, this could not be carried far enough to produce economy. It would seem, therefore, that when the author says that the advantage of two-way reinforcement is maximum when the slab is square, he should say that the disadvantage is a minimum. The most desirable form of two-way reinforcement would

Mr. Jonson.

seem to be the so-called flat-slab construction which gives a flat ceiling and requires less form work.

In looking into this problem, the writer finds that the entire theory of reinforced concrete is rather crude, probably owing to the circumstance that, when it was worked out, the properties of concrete were not sufficiently well known to make a reasonably correct theory possible. The theory of flexure of concrete is especially in need of a thorough revision. The present theory is merely an adaptation of the theory of flexure of elastic solids to a material which is subject to permanent deformations several hundred per cent. greater than its elastic deformations. Time has not permitted the writer to make such a revision in the present discussion, but he has attempted to indicate its general direction. Before such a revision can be completed, more exact determination must be made of the physical properties of concrete, *viz.*, the true modulus of elasticity, the non-elastic deformation, Poisson's ratio, initial shrinkage during hardening, and subsequent expansion and shrinkage due to variations in absorbed water. In determining these values, the influences of proportions of mixture, wetness of mix, kind, size, and grading of aggregate, age of concrete, degree of wetness during hardening, time of application of load, horizontal or vertical direction of stress, previous loading, etc., must be taken into account.

The great influence which the shrinkage has on the flexure of a reinforced concrete beam calls for the following revision of the formulas. Unless it is immersed in water, concrete shrinks during hardening. This shrinkage may amount to 0.05 of 1 per cent. When concrete is saturated with water, it again expands. This shrinkage, m, has considerable influence both on the resistance and the deflection of a beam. If a concrete beam reinforced in the bottom only is supported by the forms until all the shrinkage has taken place, either the concrete will be in tension and the steel in compression, or the concrete will crack. It is on the safe side to assume the latter. If, then, the forms are lowered slightly, but not enough to cause the beam to carry its own weight, the cracks will close at the top of the beam. When the forms are removed entirely, the concrete in the top of the beam will compress, so as to allow a part of the crack to close until the compressive stress developed in the closed part of the crack multiplied by its leverage, $d\ j$, about the center of the reinforcement is equal to the bending moment produced by the load. It may be seen from the strain diagram (Fig. 3) that:

$$\frac{\frac{f_c n}{E}}{\frac{f_s}{E} + m} = \frac{k}{1 - k} \qquad (1)$$

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and that, therefore,

$$k = \frac{f_c\, n}{f_s + f_c\, n + E\, m} \quad \ldots\ldots (2)$$

and, also, that

$$\frac{f_c\, k}{2} = f_s\, p \quad \ldots\ldots (3)$$

$$f_c = \frac{2\, f_s\, p}{k} \quad \ldots\ldots (4)$$

p being the steel ratio, which, therefore, is

$$p = \frac{f_c\, k}{2\, f_s} \quad \ldots\ldots (5)$$

Inserting this value of f_c in Equation (1), we have

$$\frac{2\, f_s\, n\, p}{k\,(f_s + E\, m)} = \frac{k}{1 - k} \quad \ldots\ldots (6)$$

$$k = \sqrt{\left(\frac{f_s\, n\, p}{f_s + E\, m}\right)^2 + \frac{2 f_s\, n\, p}{f_s + E\, m}} - \frac{f_s\, n\, p}{f_s + E\, m} \quad \ldots\ldots (7)$$

and also from Equation (6), that

$$f_s = \frac{E\, k^2\, m}{2\, n\, p\, (1 - k) - k^2} \quad \ldots\ldots (8)$$

$$f_c = \frac{2\, E\, k\, m\, p}{2\, n\, p\, (1 - k) - k^2} \quad \ldots\ldots (9)$$

From Fig. 3 it will also be seen that the reacting moment is

$$M = f_s\, b\, d^2\, j\, p = f_c\, \frac{b\, d^2\, j\, k}{2} \quad \ldots\ldots (10)$$

With regard to the maximum moment on a beam, the values of f_s and f_c are fixed, and from these follow those of j, k, and p. In 1:2:4 concrete, the value of $E\, m$ may vary from — 3 000 to 15 000, and that of n from 10 to 30. Hence, if the working stresses are 16 000 for steel and 650 for concrete, the steel ratio, p, may vary between the following limits:

$E\, m$	n	k	p
—3 000	10	0.333	0.675%
—3 000	30	0.600	1.218%
15 000	10	0.174	0.355%
15 000	30	0.386	0.784%

The greatest steel ratio is required, therefore, when the concrete is kept so wet that it expands slightly during hardening instead of shrinking. All concrete which is allowed to harden in air and finally dry out

Mr. Jonson.

shrinks more or less, according to the rapidity with which it is allowed to dry. Tests should be made to determine the minimum shrinkage which may be definitely counted on. In the meantime, 3 000 may be assumed as a safe minimum value for $E\,m$, which makes:

$E\,m$	n	k	p
3 000	10	0.255	0.52%
3 000	30	0.507	1.03%

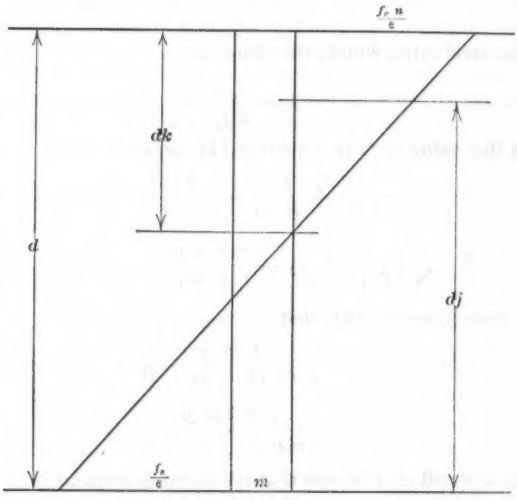

FIG. 3.

The value of $j\,p$ then varies between the limits:

$E\,m$	n	p	j	$j\,p$
—3 000	10	1.22%	0.889	1.086%
—3 000	30	1.22%	0.800	0.977%
+3 000	10	1.03%	0.915	0.944%
+3 000	30	1.03%	0.831	0.857%
15 000	10	1.22%	0.942	1.150%
15 000	30	1.22%	0.871	1.063%
15 000	10	1.03%	0.942	0.972%
15 000	30	1.03%	0.871	0.897%

This shows that for each steel ratio the strength of a beam increases slightly as it shrinks, but the effect is not marked, the main effect of the shrinkage being felt in determining the steel ratio. The strength also decreases slightly as the permanent set increases, but here, also, the main effect is on the steel ratio. A beam figured by the current

Mr. Jonson.

method would require a steel ratio of 0.77% instead of from 1.03 to 1.22%, as it should be, and would give a resistance, jp, of 0.67%, whereas, if figured by the correct formula, the resistance is only 0.62% for a beam kept wet and 0.64% for one hardened under ordinary conditions.

If the correct steel ratio had been used, the strength would have been increased in proportion. In other words, the current method exaggerates the stress on the concrete. Instead of 650 lb. per sq. in., as assumed, there is only 525 lb. It is natural that the continued yielding should reduce the unit stress by increasing the distance to the neutral axis. The higher unit stress which exists for a few hours after applying the load can do no harm, because it is relieved by the yielding of the concrete. On the other hand, the stress on the steel is, of course, correspondingly under-estimated, being actually 16 800 instead of 16 000 as figured.

Tests made by Mr. E. B. Smith, of the U. S. Office of Public Roads, have shown that 1:2:4 concrete stressed to 700 lb. per sq. in. had a modulus of deformation of 2 800 000 lb. per sq. in. immediately after the load was applied, but that this modulus changed its value as follows: in 1 hour to 2 600 000; in 24 hours to 2 000 000; and in 21 days to 1 400 000. A test made by Mr. C. M. Montgomery* in the Laboratory of the New York Board of Water Supply, confirmed these results in a general way. This test was made on a cylinder, 8 in. in diameter and 16 in. high, of about 1:2:4 mix, several years old, and thoroughly air-dried. The unit load was 300 lb. per sq. in. The modulus of elasticity maintained a constant value of about 3 000 000 ($n = 10$), but the total deformation did not become constant until the load had been on for 10 days. The modulus of total deformation, F, varied as follows:

	F.	n.
Initial	3 000 000	10
After 17 hours	1 800 000	17
After 2 days	1 600 000	19
After 4 days	1 300 000	23
After 7 days	1 200 000	25
After 10 days	1 100 000	27

Mr. F. R. McMillan, of the University of Minnesota, in his paper on "Shrinkage and Time Effects in Reinforced Concrete", gives further confirmation of these facts.

The deflection of a beam is due to the curvature which results from the stretching of the fibers on one side of the neutral axis and the corresponding compression of those on the other side. The angle of

* See Mr. Montgomery's discussion, p. 1712.

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inclination thus produced in a very small part of the length, dx, is, therefore,

$$\frac{f_c\, n + f_s + E\, m}{E\, d}\, d\, x \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

and, hence, if x is taken as the distance to the free end of a cantilever, the deflection is

$$D = \int_o^l \frac{f_c\, n + f_s + E\, m}{E\, d} \times d\, x \ldots\ldots\ldots\ldots (12)$$

The moment on the beam may be expressed in terms of k, as follows:

$$M = f_s\, b\, d^2\, j\, p = \frac{E\, b\, d^2\, k\, m\, p \left(1 - \frac{k}{3}\right)}{2\, n\, p\, (1 - k) - k^2} \ldots\ldots\ldots\ldots (13)$$

If a diagram is constructed in which the values of k are the abscissas, and the ordinates of two curves, respectively, represent the corresponding values of

$$\frac{M}{E\, b\, d^2\, m}, \text{ and } f_c\, n + f_s + E\, m,$$

the value of the latter function which corresponds to a known value of the former may thus be found in a practical manner. If these values are utilized as ordinates in constructing a diagram on the beam, similar to an ordinary moment diagram, the summation required by Equation (12) may be accomplished by treating this diagram as if it were a diagram of a distributed load, the moment of which is wanted about the free end. The free end ordinate thus obtained is equal to

$$\int_o^l (f_c\, n + f_s + E\, m) \times d\, x \ldots\ldots\ldots\ldots\ldots (14)$$

In this way the deflection of any beam subject to statically determinate moments may be found. The middle deflection of a beam supported at both ends is found by treating each half as if it were a cantilever and taking the average of the deflections of these. The deflection of any point may be found by this method, if the two end deflections are plotted and then the proportionate deflection found at the dividing point.

The curve of flexure may be constructed by a graphic solution of the equation

$$y = \int_x^l \frac{f_c\, n + f_s + E\, m}{E\, d} (x - x_1)\, d\, x \ldots\ldots\ldots\ldots (15)$$

x_1 being the abscissa corresponding to y.

The calculation of a statically indeterminate construction by the foregoing method would be exceedingly laborious, but, fortunately,

Mr. Jonson.

such forms of construction are very rare in reinforced concrete. Most cases which are treated as indeterminate are not indeterminate at all. Take, for instance, the case of a continuous floor girder extending from one side of a building to the other, a total length of 60 ft., and supported by two columns each 5 ft. from the center. At a point 4 ft. from the column, the reinforcement is carried down from the top to the bottom of the girder. On one side of this point only positive moments are resisted and on the other only negative ones; hence, at the point where the reinforcement crosses the center line, the moment must be zero. The correct method of calculating such a girder is much simpler than by the illegitimate use of the theorem of three or of four moments.* Construct a static moment diagram on each span as if it were discontinuous. Then divide the diagram of the long span by a straight line in such a way that the positive moment at the middle of the suspended span is numerically equal to the negative moment in the double cantilever over the column. Then draw a horizontal line representing the negative moment across the middle span, and the area included between this line and the first moment curve will be the true moment diagram. The point at which the first straight line intersects the moment curve of the side span is, of course, the point at which the reinforcement should cross the center line. Likewise, any other case may be resolved into a system of cantilevers and suspended spans. The only truly indeterminate cases are those in which both the top and the bottom of the beam are reinforced. This is a form of construction which should be avoided on account of the difficulty of calculating it correctly. For the same reason, slabs reinforced in two directions should also generally be avoided, unless the slab can be made square and the problem thus simplified.

When the foregoing observations are applied to the author's problem, it resolves itself into the simpler problem of calculating a cantilever slab extending from each side of the girders and supporting a central suspended panel. If the edges were securely fixed, the projection of the cantilever might be made about 15% of the span, but if the torsional resistance of the girder has to be depended on to carry the live load of one panel when the adjoining panels are not loaded, it would probably have to be less. We have then the problem of finding such a value of the author's coefficient, α, and of a coefficient, β, as shall make the center deflection of the slab the same when figured longitudinally as when figured transversely. When the slab is supported on deflecting girders there enters into the problem also a coefficient, β, representing the percentage of the load which is carried by

* The theorem of four moments is the general form which applies to all cases; the theorem of three moments is applicable only when the two middle moments are of the same magnitude, which they generally are not when the girder is stiffly connected to the columns. See "The Theory of Frameworks with Rectangular Panels," by Ernst F. Jonson, Assoc. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LV., p. 413.

Mr. Jonson.

the beam at its point of support on the girder. The reinforcement in the other direction may carry a part of the live load at this point. This coefficient, β, should be found before α by assuming values of α and β, and then figuring the corresponding deflections of the girder and the beam next to it, which must be equal if the assumptions are correct. In figuring the deflections for the purpose of finding the transverse moments, the longitudinal girders should carry a balanced live load, but the transverse ones should carry no live load on the panel on the other side. In this case, not only the deflection of the girder must be taken into account, but also that of the edge of the cantilever slab due to the twisting of the girder which results from the absence of live load on one side. When figuring with the α and β which give the greatest longitudinal stress, the conditions, of course, are reversed.

It is evident that a direct solution of this problem is impossible, for it is necessary to start with assumed values of α and β, which must be corrected if the deflections do not turn out to be equal in both directions. The influence of lateral strain may be taken account of by adding an imaginary negative load of such magnitude that the strain which it would produce in the concrete is equal to the lateral strain due to the total loading (including the imaginary negative load) on the beam in the other direction; but, in figuring the deflection due to this load, the strain in the reinforced side must be ignored, for Poisson's ratio cannot be applied to the tension side. A reasonably true solution of this problem is thus seen to be very laborious, but if it is worth while solving at all it is worth doing it right. Tables and diagrams may be constructed for practical use.

The initial stress in a reinforced concrete column due to shrinkage is as follows, f_i being the tension in the concrete and f_r the compression in the steel:

$$f_i = f_r p$$

$$\frac{f_i n}{E} + \frac{f_r}{E} = m$$

$$f_r = \frac{E m}{p n + 1}$$

$$f_i = \frac{E m p}{p n + 1}$$

These stresses must be added to those due to load and wind. Owing to the general predominance of compressive stress, it makes no practical difference whether or not the concrete cracks. In a column with $p = 4$, and 400 lb. per sq. in. on the concrete—as favorable a case as may be found in current practice—the stress in the steel is

$$f_s = 6\,800 + 8\,700 = 15\,500.$$

Mr. Jonson.

If, on the other hand, $p = 0.5$, and the load on the concrete is 500 lb. per sq. in., then

$$f_s = 13\,000 + 14\,000 = 27\,000.$$

This means that, with the current method of figuring, unless a considerable percentage of reinforcement or a low working stress on the concrete is used, the steel is stressed nearly to its elastic limit. On the other hand, the concrete is not stressed very high, such stresses, corresponding to the foregoing cases, being, respectively,

$$f_c = 290 - 272 = -\ 18$$
$$f_c = 468 - \ 65 = \ 403$$

The correct method of calculating a reinforced concrete column, therefore, would be one which takes account of the shrinkage stresses That is,

$$f_s = f_y + \frac{E\,m}{p\,n + 1}$$

$$f_x\,n = f_y$$

f_x being the available working stress on the concrete and f_y that on the steel. As this formula gives f_y its minimum value for the maximum value of m, $E\,m$ must be given the value 15 000. If $f_s = 20\,000$, we have

$$20\,000 - \frac{15\,000}{30\,p + 1} = f_y$$

$$f_x = \frac{f_y}{n}$$

$$f_c = f_x - \frac{E\,m\,p}{p\,n + 1}.$$

We get then for extreme values of p:

p	f_y	f_x	f_c
4%	13 200	440	167
½%	7 000	233	168

which shows that the concrete is never stressed high in columns, and, for this reason, a rather high stress may be used on the steel.

In working out the foregoing discussion the writer was largely led away from the special problem of Mr. Janni's paper by the discovery that the generally accepted assumptions of reinforced concrete theory do not correspond with the facts. The solution of the problems raised by this discovery have been indicated in the foregoing discussion in a general way only. The fuller working out of the theory of reinforced concrete in accordance with the facts will be made the subject of a future paper. The foregoing discussion must not therefore be taken as the writer's well established and final conclusions, but rather as a step toward the elimination of certain fundamental errors in the theory of reinforced concrete.

Mr. Montgomery.

CHARLES M. MONTGOMERY,* ESQ. (by letter).—The following is a record of a test made in the Laboratory of the New York Board of Water Supply, in connection with the discussion of Mr. Janni's paper by Mr. Ernst F. Jonson. The specimen was a cylinder, 8 in. in diameter and 16 in. high, made at one of the tunnels of the Catskill Aqueduct, from concrete used for lining the tunnel, the mix of which was approximately 1:2:4. The specimen was made about 4 years ago, and was stored in damp sand until May, 1915, since which time it has been exposed to the air. The coarse aggregate was Fordham gneiss of 2 in. maximum size; the fine aggregate was also Fordham gneiss. The crushing strength was 3 550 lb. per sq. in.

TABLE 1.

Total load, in pounds.	Micrometer.			Deformation.	Time.	
	Front.	Rear.	Average.		Date.	Hour.
500	0.2059	0.1060	0.1556	.....	3/22	4 P. M.
1 500	0.2053	0.1032	0.1542	0.0014		
500	0.2046	0.1057	0.1551	0.0005	3/23	9 A. M.
1 500	0.2041	0.1024	0.1532	0.0024		
500	0.2048	0.1034	0.1541	0.0015	3/24	9 A. M.
1 500	0.2047	0.1010	0.1528	0.0028		
500	0.2036	0.1044	0.1540	0.0016	3/25	9 A. M.
1 500	0.2032	0.1009	0.1521	0.0035		
500	0.2039	0.1030	0.1534	0.0022	3/27	9 A. M.
1 500	0.2039	0.1007	0.1523	0.0033		
500	0.2043	0.1033	0.1538	0.0018	3/28	9 A. M.
1 500	0.2038	0.1007	0.1522	0.0034		
500	0.2040	0.1027	0.1533	0.0023	3/29	9 A. M.
1 500	0.2035	0.1004	0.1519	0.0037		
500	0.2040	0.1025	0.1532	0.0024	3/30	9 A. M.
1 500	0.2034	0.1003	0.1518	0.0038		
500	0.2035	0.1021	0.1528	0.0028	3/31	9 A. M.
1 500	0.2032	0.0998	0.1515	0.0041		
500	0.2036	0.1022	0.1529	0.0027	4/1	9 A. M.
1 500	0.2032	0.0999	0.1515	0.0041		
500	0.2040	0.1023	0.1531	0.0025	4/3	9 A. M.
1 500	0.2034	0.0998	0.1516	0.0040		
500	0.2037	0.1022	0.1529	0.0027	4/4	9 A. M.
1 500	0.2032	0.0997	0.1514	0.0042		
500	0.2037	0.1022	0.1529	0.0027	4/5	9 A. M.
1 500	0.2032	0.0997	0.1514	0.0042		

A narrow iron collar was bolted around each end of the specimen (with a measuring point projecting on each side). The distance from center to center of collars was 15 in. This specimen was not made for the purpose of determining deformations, and was therefore deficient in length. The nearness of the collars to the ends undoubtedly reduced the deformation, so that the modulus of elasticity given by this test is probably somewhat high. The purpose, however, was not to establish the value of the modulus of elasticity, but to discover to what extent deformations would continue during

* Inspector, Board of Water Supply, New York City.

a prolonged application of the load. The deformations were measured by two inside micrometers, reading to 0.0001 in., which remained in place during the entire period of the test. The contact between the micrometer and the upper measuring point was indicated by an electric buzzer. The initial load was 10, and the final load 300 lb. per sq. in. Table 1 is a record of the test. Mr. Montgomery.

C. A. P. TURNER,* M. AM. SOC. C. E. (by letter).—This paper undertakes to treat the flat plate or slab from the standpoint of the theory of work. It makes the work dependent on Poisson's ratio and moments in selected strips across the center portion of that plate. Mr. Turner.

It is fundamental to the theory that the amount of work done depends on the deflection. Now, a formula which involves work must be such that the deflection can be computed and checked with observed deflection. Otherwise, there is something wrong with the formula, but the author has apparently made no attempt to check his theory by observation. Therefore, it lacks confirmation.

Suppose we compare a circular plate supported at its center with a double cantilever beam supported at the middle. The resisting moment in the cantilever causes stresses along its length, and Poisson's ratio may be disregarded, as it has no effect on the resulting deflection, in accordance with the fundamental equations of extensional stress and strain established a generation ago and accepted by Grashof and all authorities on the subject since then. These equations are as follows:

$$Ee_1 = p_1 - Kp_2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

$$Ee_2 = p_2 - Kp_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

in which K designates Poisson's ratio, p_1 and p_2 are the external applied or apparent stresses per unit of area of cross-section of the plate, which act parallel to the axes of x and y, respectively, if these latter lie in the neutral line of the plate, and e_1 and e_2 are extensional elongations of the plate per unit of length parallel to x and y, respectively. Hence, if p_2 is zero, as it would be in a beam, $Ee_1 = p_1$, the longitudinal deformation, e_1, depends solely on p_1, and determines the deflection.

Thus it is evident that K cannot affect the internal work in a beam in a manner comparable to its effect in a slab, yet Mr. Janni proposes to introduce K into what is substantially a beam formula.

From the mathematical standpoint, Mr. Janni does not work correctly from the general fundamental equations of extensional stress and strain. In deriving his equations for the true moment of the strip parallel to the axis of x, he assumes $p_2 = 0$. In deriving his equation for the strip parallel to the axis of y, he assumes $p_1 = 0$, and, in Equations (1) and (2), if p_1 and p_2 are zero, then the Poisson

* Minneapolis, Minn.

Mr. Turner.

ratio is eliminated; and yet, having eliminated the Poisson ratio and effect, Mr. Janni arbitrarily inserts a factor involving this ratio in his equation to represent the work which, on the basis of his strip assumption, in deriving his equations for the moment on the strip, has been eliminated. The terms inserted are correct neither for the beam theory nor for the plate theory.

Consider the case of a plate: The deformations due to spherical curvature or bending are radial and circumferential. Assuming that K (Poisson's ratio) is zero, it has been demonstrated* that the work done in a circular direction is equal to that done in a radial direction. Now, radial deformations alone determine the vertical position of the cantilever plate, and though the circumferential deformations necessarily accompany radial deformations, they provide a means of storage of energy which is not involved in nor determines the vertical deflection. Accordingly, in the circular cantilever plate, half the work is done in a manner which produces no deflection. In the slab supported on separated posts, we have a cantilever area about the post, and a suspended span, which is really an inverted plate similar to the cantilever portion about the column, located about the diagonal center of the span so that treatment of the plain cantilever serves for the treatment of the combination of the cantilever and suspended span in a diagonal direction, approximately.

We will now proceed to compare the double cantilever beam and the circular cantilever plate, assuming that the plate and beam have the same thickness, and the same metal, and that each stores an equal quantity of energy, Q; in which case:

Let W_1 = the load on the cantilever beam uniformly distributed;
W_2 = the load on the cantilever circular plate;
D_1 = the mean deflection of the load in the cantilever beam;
and D_2 = the mean deflection of the load on the cantilever plate.

Since, in the cantilever plate supported at the center, half the work is done in a manner which produces no deflection, as compared with a cantilever beam, the deflection would be reduced one-half. Now,

$$Q = \frac{1}{2} W_1 D_1 = \frac{1}{2} W_2 D_2;$$

but, the amount of energy stored being assumed as the same, W_2 must equal 2 W_1, since $D_2 = \frac{1}{2} D_1$. Now, if we assume the deflection the same, it is apparent that W_2 must equal 4 W_1; or, the work of deformation is four times as great in the double cantilever beam, strained in one direction, as it is in a circumferential plate, strained

* "Concrete-Steel Construction", by Eddy and Turner, pp. 137-138.

in two directions, for equal loads; and this is on the basis that there is no Poisson action, that there is no reduction of strain per unit of stress in one direction by another stress acting at right angles thereto. Mr. Turner.

Compare a reinforced concrete beam and reinforced concrete plate on posts with square panels, taking an interior panel uniformly loaded, for simplicity, to present the essential facts: The experimentally determined formula for deflection, for the old-style, mushroom, flat plate of concrete, may be written for a square panel as follows:

$$\Delta = \frac{W L^3}{2 \sqrt{2} \times 7\,000 A_s h^2} = \frac{W L^3}{19\,800 A_s h^2} \text{ (approximately).}$$

In this case, L is the diagonal of the panel and Δ is the deflection at the diagonal center. A_s is the steel area at mid-span and h is the distance from the center of the steel to the top of the concrete.

A similar formula, worked out from a number of experiments on thoroughly cured, continuous, concrete beams, integral with the slab, these beams being about 12 in. wide and from 18 to 24 in. deep, with a 6- to 8-in. slab and from 15 to 24 ft. from center to center, is:

$$\Delta = \frac{W L^3}{5\,000 A_s h^2};$$

in which A_s is the cross-section of the steel at mid-span and h is the distance from the center of the steel to the top of the concrete.

Comparing these formulas, it will be seen that the diagonal deflection of the slab is substantially one-quarter of that of the continuous beam at mid-span, a result in keeping closely with the theory of work above outlined, except that the slab, being integral with the beam, reduces its deflection.

For simple beams, the writer deduced, and published in 1909, a formula in which the coefficient is taken as the reciprocal of 850. Now, the relative stiffness of the continuous and simple beam is known by theory to be as five to one, so that, from these considerations of numerous experimental results, it would appear that the slab integral with the continuous beam has increased its stiffness approximately 18 per cent. The Poisson effect which Eddy finds in his theory is a little greater than this, increasing the stiffness 33 per cent. The simple beams experimented on did not have true knife-edge supports, and this discrepancy would be expected for that reason.

The question, "what is the nature of the Poisson effect in reinforced concrete"?, may be here considered.

Reinforced concrete is not a homogeneous material, but a composite of steel and concrete combined. There can be no molecular

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action, such as is found in a homogeneous material. The lines of stress, however, may be represented somewhat as are the variations of shade and shadow in a copper half-tone plate. The picture is thrown on the plate through a screen. Shade and shadow are not continuous but are interrupted, and, if the interruptions are sufficiently fine-grained, the appearance to the eye and the general effect is that of the photograph.

So with reinforcement, if we are to imitate homogeneous material, we must have the reinforcement distributed so that the resulting structure is relatively fine-grained. It is on this principle that the imitation of a homogeneous plate may be made. The distribution of the stress and the manner in which the stress in one direction may co-act with that in another has been discussed at length.* The object of this short discussion is to point out the theoretical error involved in the supposition that work, where there is double curvature, is measured largely by the Poisson effect. Poisson effect or action enters into the problem as a relatively minor factor, a matter which Mr. Janni has not discovered, because he has tried to compare beam strips and work through this Poisson coefficient and get at the external work, something which is not possible in working from strips, as the writer pointed out from comparison of the double linear cantilever and the circumferential cantilever.

Much of the difference of opinion among the Engineering Profession to-day regarding beam action and imitation of plate action in concrete may be accounted for through the failure of the Profession generally to understand the theory of work in its relation to horizontal shearing rigidity and flange rigidity.

If we have two planks, for example, placed one upon the other, and load them, the stiffness and strength is substantially the aggregate of the two planks, because the friction between them would ordinarily be very small, and the upper corner of the lower plank would slide by the lower corner of the upper plank, if they were of the same length. Now, by bolting and gluing these planks rigidly together, we increase the stiffness fourfold and the strength twofold, because we prevent this sliding one upon the other. The result produced by the resistance of this shearing rigidity at the neutral plane of the beam or slab becomes apparent.

If we test a slab which, though hard on the outside is not thoroughly cured at the neutral plane, there is no proportionality at all between the flange stresses and the deflection. If, however, we allow the slab to become thoroughly cured, and compare work which is thoroughly hardened and rigid throughout, then, under these circumstances, we find the most gratifying concordance between relative deflections and stresses.

* "Concrete-Steel Construction", by Eddy and Turner.

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Initial loading, however, involves some allowance for the shrinkage and temperature stress of hardening. If we are to measure stresses in the concrete of slabs and beams when first loaded, even though they are thoroughly cured, the deformations in the concrete compared should be those of the recovery after the initial loading, and not the initial deformation which includes the effect of the load plus that of temperature and shrinkage stresses in hardening. In subsequent loadings, or after one or two repeated loadings, almost perfect elasticity will be found, but the effect of shrinkage and temperature stresses is more local than general in the slab, and the deflections and steel stresses can be computed accurately with ordinary concrete executed with ordinary care when the work is thoroughly cured and hardened.

Much conscientious hard work in investigating concrete is of no value whatever, because investigators have failed to appreciate the fact that cured concrete is the only kind on which it is worth while to make measurements, because it is the only kind of concrete in which there is a definite relation between the deflection and the amount of load, and consequently the only kind to which mathematical theory of any kind is applicable. Just as sure as concrete is only partly cured along the neutral plane, although hard on the outside, there is no relation between deflection and stress which we have any theory capable of computing, and our measurements then are of no value except in showing what may occur in a slab at this exact stage of hardening, and to duplicate this stage or know that two concrete specimens have reached exactly the same stage of partial curing is something no one has been able to do as yet; hence the measurements made are of academic value only.

Another matter which is very puzzling to many is the fact that the mechanics of a solid is such that ordinary rules of statics do not apply, that is, the rules of statics as applied to the separate members of framed structures are not here applicable. For example, under pure statics, a force in one direction cannot affect the magnitude of the force at right angles thereto. In other words, it has no component which affects it according to statics; but, in the mechanics of a solid, that is not the case. A deformation in one direction under a given force affects and influences the deformation produced by a force at right angles thereto, and it may thus reduce the work done by the external forces to the surprising amount of 80 per cent.

Failure to understand these elementary relations has caused many building departments to adopt irrational regulations, thereby placing a premium on more dangerous types of construction and putting at a disadvantage the more rational and safer types of design.

The difference between beam resistance and slab resistance is, in part, that in the beam horizontal shearing stresses act only in horizontal planes, but in the slab, with its double change in curvature, horizontal

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shearing forces act in vertical planes as well. Now, the action of these latter forces is disregarded in any beam strip theory, a fact recognized in the classical theory of plates for fifty years. Early flat-plate theories did not take into consideration the Poisson effect at all, a fact which Mr. Janni and many present-day engineers apparently have overlooked.

Mr. Janni assumes that putting an arbitrary factor involving Poisson's ratio into a beam formula makes it apply to a plate. The magnitude of this error may be found by computing the deflection by his theory and comparing the computed values with the experimentally determined deflection.

Working on the basis of beam theory in the design of flat slabs, the quantity of steel used frequently overbalances the strength of the concrete, and unsatisfactory work frequently results. The comprehensive theory, which takes into consideration properly the double curvature and twisting of the plate, whether it is supported on four sides or on a series of posts, will insure the avoidance of this undesirable kind of design. The idea that the Poisson effect in a reinforced concrete slab is a property either of the concrete or of the steel is erroneous. The mathematical coefficient applies to true molecular phenomena. The imitation of that phenomenon in a fine-grained composite structure has no direct relation to a plate, either of uniform concrete or of homogeneous steel.

Another error, which it appears to the writer has crept into many discussions, is the idea that Poisson's ratio is a constant. Its value would seem to depend entirely on the shape of the piece tested, and be a constant for the same kind and shape of test piece, but not for different kinds and different shapes. A square bar would give one value; a wide rectangular bar would give two values under tensile stress; a plain flat plate bent in one direction gives no criterion as to the Poisson effect in a plate bent in two directions under the fundamental equations of stress and strain.

The magnitude of the Poisson effect may be readily shown from the fundamental equations of internal stress and strain—Equations (1) and (2)—to be a maximum when $p_1 = p_2$. In other words, it requires equality of stress in the direction of x and y, respectively, for the maximum efficiency. The previous demonstration of the action of the circumferential cantilever plate is based on this equality, for when the forces along x are greater than those along y, circumferential deformations enter into the deflection largely along x. Hence, the Poisson effect, as well as the effect of double curvature, varies between square and rectangular panels according to the law of this variation. This law is found experimentally to indicate that, in a rectangular panel, the length to be considered is not a mean of the two sides, but is that of the longer side, a relation which holds good with panels in which the width is six-tenths of the longer side. The diagonal length

from column to column cannot be used in a rectangular panel, and for that reason the empirical formula for deflection published in 1909 by the writer was confined to the use of the direct span from column to column, using the longer side of the rectangular panel. Mr. Turner.

Regarding the shrinkage of concrete in columns, this, in the writer's judgment, is a matter which should be given more consideration than is usually accorded it. If the column is well tied in, 12 000 lb. per sq. in. on the steel, if it is hard, should be permissible, provided that the column bars are lapped and not butted. Lapping permits an adjustment during the curing stage which is not possible if the bars are butted. Tests show higher values where the longitudinal steel does not abut the face-plate of the testing machine.

No engineer who has loaded a plain concrete plate and witnessed its performance under a drop of temperature of 25° could rationally talk of the evanescent value of the tensile strength of concrete. In practical construction, the splice or joint eliminates this resistance; still, the deflection is increased but little, and the steel stress likewise undergoes little change. This observation and experience indicate the magnitude of the error involved in Mr. Godfrey's conception of the problem.

Mr. Godfrey has stated that "Tests ought to be made by the Federal Government, not by patent owners, interpreted by their hired experts." His contention, however, is fallacious, and this is shown clearly by the fact that the test of the Curran Building, in St. Louis, was conducted by Robert W. Hunt and Company, and that the report of that company cannot be regarded as unworthy of credence simply because it was made by paid experts. Mr. Godfrey should be informed that numerous articles in the technical press indicate the great extent to which deck mushroom floors have been adopted, in bridge work with columns from 30 to 40 ft. in length, and spans from 30 to nearly 50 ft. There are also many descriptions of tests of such floors; for instance, the floors of the Minneapolis Post Office were tested by the Federal Government.

As a result of the writer's observations, he is firmly convinced that in the past many engineers have attributed the increased rigidity of the work to the tensile strength of the concrete, instead of ascribing it to the increase in horizontal shearing rigidity along the neutral plane of the slab due to thorough curing. Satisfactory evidence of the truth of this fact may be readily brought out by testing a slab without reinforcement. The great importance of this shearing rigidity has been illustrated by the example of the two planks where it increases the stiffness fourfold and the strength twofold. Permitting horizontal shearing rigidity to become a factor in vertical planes has likewise a phenomenal effect on the resistance of the plate, an effect which is brought out only by the double curvature.

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It may be argued that the concrete matrix binds the steel and furnishes this shearing rigidity to a large extent. The error of this view will be obvious from the consideration of the example of the two planks which do not furnish the shearing rigidity of the bolts or the glue which increase the stiffness fourfold. Neither does the steel by itself nor the concrete by itself furnish the shearing rigidity of the composite structure. The action as a composite structure is due to bond stress, and bond stress is a shearing resistance added to the combination by the steel embedment and the shrinkage of the concrete in hardening about the steel. Thus, where steel is embedded in concrete we are dealing with a new element which has not been given proper consideration by those who fallaciously assume that the effect of the addition of this shearing rigidity is a property of the concrete. Were this a fact, reinforcement would be abandoned. If concrete can furnish three or four times the tensile resistance of the steel, wherein lies the utility of the steel? In order to account for slab phenomena, an explanation based on an unheard of tensile strength of concrete is offered, the absurdity of which becomes apparent only on analyzing the effect of horizontal shearing forces in horizontal planes and horizontal shearing forces in vertical planes. The erroneous idea that a mere Poisson coefficient accounts in fact for the great difference in performance of the slab and beam becomes evident when we reduce this coefficient to zero and find that the work of deformation for the same load is one-fourth as great with the circular cantilever plate as it is with the linear cantilever plate. No more forcible illustration of the action of the horizontal forces in vertical planes can be brought out than this explanation.

No more cogent reason can be offered for abandoning beam strip theory than the concordance of true theory with empirical, experimentally determined and proved, formulas for deflection.

Distribution of Stress.—The composite structure not only differs from the homogeneous plate in respect to the difference between it and the infinitesimally fine grain of the make-up of a homogeneous plate, but also differs in the distribution of resistance under the known law of the bond shear or bond stress. The action of these forces is most efficient where the horizontal shearing deformation is a maximum. This occurs toward the end of the cantilever on each side of the line of inflection in the flat slab supported on columns. Thus, there are differences between distribution of stress in the homogeneous plate and the imitation of the homogeneous plate.

The unique application of the mathematical flat-plate theory of Eddy is based on the assumption of an imaginary solid endowed with those imaginary properties which will produce the deflection of the composite plate. By this expedient Eddy succeeds in figuring the vertical geometry of the composite plate under load, that is, the

vertical geometry of the imaginary plate which fits or substantially agrees with that of the composite plate. Having determined this geometry, he has determined mathematically the geometry of deformation, and, reducing this geometry of deformation to stress, he is able to compute, with a high degree of precision, steel stresses and deflections in all recorded tests on thoroughly cured concrete plates. Thus, in one and the same theory, Eddy alone has succeeded in deriving equations by which the deflection and steel elongations are admitted by all to be computed in keeping with results of experiment. It seems to the writer that all criticism of Eddy's methods is based on an entire misapprehension of the stages by which they are derived, and a misunderstanding of the theory of work which, in its simplest and most elementary form, the writer has endeavored to present in the foregoing discussion. Mr. Turner.

Even in the reinforced concrete beam, it is too commonly assumed that there is a much closer analogy to homogeneous material than actually exists. In the reinforced concrete beam, horizontal shears in vertical planes are also to be found which do not exist in the homogeneous beam. Consequently, these shears which in the lower plane cause greater deformations close to the surface of the steel than between the bars, affect the amount of resistance required to hold the external loads in equilibrium at the center. Since it may be shown that the internal work of deformation in view of the bond stress is in part lateral, the energy there stored has a marked effect in reducing the deflection and the work of deformation, all erroneously attributed heretofore to an overestimated or imaginary direct tensile strength of the concrete. When the work of indirect stress is followed out, and a correct theory of the beam is presented, much of the difference of opinion among engineers on reinforced concrete will disappear.

Time Effect.

Rich Concrete.—Concrete grows more rigid with time, aside from shrinkage and temperature effects, but 90-day concrete of a 1:2:4 mix cured at a temperature greater than 65° shows little increase in deformation where the stress is not more than one-third of the ultimate. For greater stresses it shows a considerable and gradual increase in deformation with continuance of pressure, up to a certain limit, which must be considered in the correct interpretation of test results.

Lean Concrete.—In discussing time effects which occur during the curing of concrete, the properties of a standard 1:2:4, or rich, concrete should not be confused with those of a poorer concrete. In cases where, through ignorance or otherwise, buildings have been erected in which a concrete has been used consisting of a 1 cement to 6 bank run of gravel, when such a mix was no better than 1 cement to 5 sand, the quantity of cement is insufficient to fill the voids, and the concrete is

Mr. Turner. not solid. Its physical properties are then very different from that of a standard 1:2:4 mix. Such concrete will continue to deform with time to a surprising extent, without showing many noticeable cracks, and the process of sagging will continue apparently without definite limit for a period of several years. It is evident that such concrete should be avoided as wholly unfit for building purposes, as its characteristics are entirely different from those exhibited by a suitable mix, with which it should not be confused.

Mr. Eddy. HENRY T. EDDY,* ESQ. (by letter).—*1.—Fundamental Equations.*—In case the rectangular axes of x and y lie in the horizontal neutral plane, and z denotes the deflection of a flat slab or plate, it is known that the correct moment equations are†

$$\left.\begin{aligned}(1-K^2)\,M_1 &= \pm E\,I\left(\frac{d^2z}{dx^2}+K\frac{d^2z}{dy^2}\right)\\ (1-K^2)\;M_2 &= \pm E\,I\left(\frac{d^2z}{dy^2}+K\frac{d^2z}{dx^2}\right)\end{aligned}\right\}\quad\ldots\ldots\ldots(1)$$

in which, for convenience, K is used to express Poisson's ratio of lateral deformation, in place of the expression $\frac{1}{m}$, used by Mr. Janni, M_1 and M_2 are the apparent bending moments per unit of width of slab due to the applied forces or loads which tend to cause flexure of lines initially straight and drawn on the slab parallel to the axes of x and y, respectively.

These equations are substantially the same as those used by Grashof in his classical German treatise on the theory of elasticity and strength (1878), and copied in Lanza's American treatise on applied mechanics, which was used for many years, since about 1885, as a textbook at the Massachusetts Institute of Technology. They have been accepted universally as valid, and have never been disputed by any competent writer.

These equations are established by expressing the conditions for the statical equilibrium of an infinitesimal rectangular element of a plate or slab supporting an element of load and subjected on its vertical sides to vertical shearing stresses, and to the tensions and compressions arising from the bending moments as well as to the horizontal shears due to twisting moments. These comprise all the forces that can possibly act on the element, and the conditions for equilibrium are the sufficient and complete expressions for the exact solution of the problem in hand. Any other attempt less general than this must of necessity be, at most, more or less of a tentative approximation, if indeed it can be designated as any approximation

* Minneapolis, Minn.

† See "Concrete-Steel Construction", Eddy and Turner, Minneapolis, 1914, p. 173.

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at all. So far as known, no such attempt has had any success in approximating to the actual stresses or deflections shown by tests; and the solution which has been derived from Equations (1) has been found to be in numerical agreement with all published tests and many others as yet not published.

Now, the fundamental equations used by Mr. Janni differ from these in reality only in one point, namely, by suppressing and leaving out the last term of each of them. That this is the fact, however, is not entirely apparent, at first view, because Mr. Janni in his mathematical work has chosen to develop the subject by attempting to modify and apply the equations of Professor Fraenkel for beams, which express the work performed during deflection by an assumed auxiliary force applied at the point where the deflection is measured. It may be readily shown, however, that Fraenkel's equations are equivalent to the ordinary equation of flexure of a beam as quoted by Mr. Godfrey, namely,

$$M = E \frac{d^2 z}{d x^2}.$$

When Mr. Janni says it is known that the expression for the work is given by his Equation (2), the writer feels compelled to say that the work in a slab or plate is not known to be expressed correctly by this Equation (2), because it is incompatible with the accepted Equations (1) given by the writer, besides other reasons which will be given later.

The only criticism that Mr. Godfrey makes of Mr. Janni's equations is that they involve such mathematical complications as to prevent most engineers from being able to judge of their correctness. He, himself, does not suggest that they are incorrect, and he is evidently not aware that they are incompatible with the ordinary equation of flexure which he quotes. His remarks, however, convey the impression that he had understood how they were established; but, since they are in fact not correct, Mr. Godfrey cannot have understood how they were derived mathematically.

It is possible to make this assertion of incompatibility between these equations with certainty, notwithstanding the fact that M in Equation (2) is the apparent moment of that part of the total load which Mr. Janni has arbitrarily taken as acting to produce M, while the M_1 in the writer's Equations (1) is due to the entire load, because Equations (1) enable us to determine the shearing stress per unit of width of the section where the bending moment is M_1, and Mr. Janni's arbitrary distribution of the total loading on each beam strip also determines the shearing stress in that strip. This last differs from that previously mentioned, therefore the systems are incompatible. What Mr. Janni has in effect done by his formulas is this:

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First, he has reduced the deflection below that of a separate beam strip by assuming the entire load on the slab to be arbitrarily apportioned between the assumed beam strips, which cross each other at right angles parallel to the edges of the panel. Leaving out of account all question as to how this would agree or disagree with some other assumption—such, for example, as diagonal beam strips—the arbitrary subdivision of the loading between the two sets of strips cannot pretend, of course, to any mathematical accuracy, and would have to give way before any mathematically exact method. There is a much more accurate investigation of this kind of hypothesis due to Danusso and edited by von Bronneck* in German, on concrete slabs with crossed reinforcement, in which the strips have identical deflections at all points of crossing, instead of merely on the center strips, as proposed by Mr. Janni. Since the beam strips of Mr. Janni are not required to have identical deflections at any other points than along the middle strips of the panel, his method is necessarily insufficient to secure accurate results.

Secondly, Mr. Janni further reduces deflections by introducing the factor, $(1 - K^2)$, arbitrarily, so far as appears from anything in the paper. It may very well be the fact that this factor is involved in the correct equations, as it is in fact in the writer's Equations (1), but to attempt to take account of the effect on any given strip of the adjacent parallel strips by the insertion of this factor is entirely unwarranted unless the correctness of the procedure is otherwise established.

However, regardless of any question of these two attempted corrections of beam theory to make it apply to the beam strip theory of slabs, Mr. Janni has misunderstood and incorrectly used Fraenkel's equations of work and deflection in several particulars, for, in the first place, his Equation (1) is incorrect, and should read

$$d\,L = \frac{1}{2} D\,d\,F$$

because $d\,L$ is the elementary amount of work performed by the elementary force, $d\,F$, when it is gradually applied with the rest of the external load during the progress of the deflection, D.

Mr. Janni, in his Equation (1), has practically assumed that $d\,F$ is already acting on the beam before the rest of the external load is applied to it, whereas it is in fact a part of that load and subject to the same conditions as any other part of it. This alteration alone would change Mr. Janni's results by 100%, were the rest of the development correct.

* Ernst and Son, Berlin, 1913.

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There is, however, another error involved in his Equation (2) by which the error in Equation (1) is nullified. This can best be appreciated by first considering the external moment, M:

Let M = the total external moment, including F, as given in Equation (4).

Let $\mathfrak{M}$ = the external moment, excluding F, as given in Equation (7).

Let $\mathfrak{m}$ = the moment of F.

Then $M = \mathfrak{M} + \mathfrak{m}$.

Now, in Equation (2), if we disregard the factor, $(1 - K^2)$, the work, L, should be that performed by F during the gradual application of the total load, but Equation (2) is written incorrectly, for it contains the factor, M^2, where it should have, in place of this, the widely different factor $M\,\mathfrak{m}$.

These errors in Equations (1) and (2) are curiously so related, the one to the other, that Equation (3) is the same as would be obtained were the alterations just pointed out made in Equations (1) and (2).

It appears, therefore, that Mr. Janni did not understand the true import of his Equations (1) and (2). Much confusion would be avoided in this paper were a careful distinction observed between the moments, M and $\mathfrak{M}$, as defined by Equations (4) and (7), since M is used in the paper for either the one or the other.

It follows, also, that Mr. Godfrey did not comprehend this matter any better, although he did not say so.

Furthermore, Mr. Janni has introduced the same two arbitrary reductions into the fundamental stress equation for beams that he had already attempted to introduce into his equations for work and deflection, for he states that "the general equation of stability given by theory" is

$$f\,\frac{I}{h} = (1 - K^2)\,M,$$

in which M is the external moment due to the arbitrary and inaccurately assumed external load on the strip, excluding F, while f is the stress in the steel and h is its distance from the neutral axis. There is, however, no proof of the fact that this equation is given by any known theory, although when $K = 0$, the equation is the correct ordinary equation of the stress, f, due to the applied moment, M.

In view of these two corrections, introduced by Mr. Janni into beam theory in his attempt to make it apply to beam strips, the question should be fully settled by him whether both these corrections are required at one and the same time, or whether he should use only one of them, and, in that case, which one is required. He could, perhaps, settle that question by comparing his results with test data. It is not probable, however, that either or both of these corrections will bring the theory into good agreement with observed

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results. Unless more secure foundations can be established for Mr. Janni's mathematical superstructure than appear in this paper, no great confidence can be put in the substantial validity and accuracy of his conclusions.

2.—Shrinkage, Time Effects, and Variation of the Modulus of Concrete with the Stress, etc.—The conclusions of Mr. Jonson are self-evidently incorrect, because it appears, from his Equations (8) and (9), that f_s and f_c, which he finds to be proportional to m, would vanish in case $m = 0$, a case which he himself admits might occur were the concrete kept wet during the process of curing. These absurd results are due to the fact that his Fig. 3 does not represent correctly the relations under consideration.

These formulas have reference to the theory and design of reinforced concrete beams, a subject which has received more careful theoretical and experimental attention than would be implied from Mr. Jonson's remarks, which also intimate that he does not regard himself as having done more than indicate the path along which a radical revision of the theory now current should proceed, a theory on which many eminent engineers have spent their very best efforts. The failure of his Equations (8) and (9) make it evident that his indications are incorrect.

The writer finds himself unable to agree in ascribing any such controlling effect on the theory of beams to shrinkage as Mr. Jonson thinks occurs.

Recourse is to be had in all such matters to scientific experiment and observation, and the object of any theory is to give a rational explanation and co-ordination of the observed facts. Perhaps the largest single body of well-ascertained data on reinforced beams is found in Technological Paper No. 2, Bureau of Standards, which reports the tests on 333 beams, 8 in. wide, 11 in. deep, 13 ft. long, and 12 ft. span, loaded at the one-third points, and with percentages of reinforcement varying from 0.5 to 2 per cent. The gauge length was 29.25 in. Four different kinds of concrete were used, cinder, gravel, broken limestone, and broken granite.

The one noticeable and unmistakable phenomenon of the last three kinds of beam is shown in the graphs in which the abscissas represent the elongations or stresses in the steel at mid span, and in which the ordinates represent either $\frac{M}{b\,d^2}$, or the load, according to the scale used. All these graphs have a pronounced knee at a unit steel stress of about 5 000 lb., and are in general shape like those found in tension tests of a steel rod at the yield point. The tests of cinder-concrete beams also yield similar results, but lack to some extent the sharply defined definite shapes of those just mentioned. Between unit stresses

of 6 000 and 30 000 lb., the graph has practically a uniform slope. The elastic limit of the steel used was from 33 000 to 41 000 lb., and its ultimate strength was between 52 000 and 64 000 lb. The working and test stresses, therefore, lay between 6 000 and 30 000 lb. The important question, therefore, is this: what is it that causes the knee at a unit stress of about 5 000 lb.? The question seems to have been answered in a paper* by Mr. Duff A. Abrams, on "Tests of Bond between Concrete and Steel," in which, besides pull-out tests of embedded rods, Mr. Abrams reports a large number of tests on reinforced beams loaded at the one-third points. He finds (page 207) that: Mr. Eddy.

"* * * bond between concrete and steel may be divided into two principal elements, adhesive resistance and sliding resistance. * * * The adhesive resistance must be overcome before sliding resistance comes into action. In other words, the two elements of bond resistance are not effective at the same time at a given point". (Page 208.) "If we conclude that adhesive resistance was overcome at the first measurable slip, it will be seen that the adhesive resistance was about 60% of the maximum bond resistance. This ratio did not vary much for a wide range of mixes, ages, size of bar, condition of storage, etc. * * * Sliding resistance reached its maximum value for plain bars of ordinary mill surface at a slip of about 0.01 in. The constancy in the amount of slip corresponding to the maximum bond resistance for a wide range of mixes, ages, size of bar, conditions of storages, etc., is a noteworthy feature of the tests. With further slip the sliding resistance decreased slowly at first, then more rapidly, until with a slip of 0.1 in. the bond resistance was about one-half its maximum value." (Page 216.) "Slip of bar was a phenomenon in all beam tests in which careful slip observations were made. * * * Slip was first observed in the middle region of the span at loads producing a tensile stress in the steel of about 6 000 lb. per sq. in. * * * As the load was increased, slip of bar progressed through the outer thirds toward the ends of the beam at a rate nearly proportional to the increase of load."

Without making further quotations from this important paper, it is clear that any initial shrinkage stresses which the concrete may exert on the steel are limited in their action by the bond, so that they have no important effect on the steel after it reaches a unit stress of 6 000 lb., and therefore they need not be considered as having any influence on working stresses after that. The knee of the graph of the steel stresses in beams is evidently reached when the adhesion ends and the slip begins in the central region, which later progresses into the end thirds, this masking any initial shrinkage effect.

In the tests of these simple beams (page 218) "the maximum bond resistance was materially increased by the additional overhang" at the ends of the beams beyond the supports. In continuous beams it

* *Bulletin No. 71*, Engineering Experiment Station, University of Illinois, 1913.

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is evident that there is no opportunity for the rods to slip at the points of inflection, as they may at the ends of simple beams, and for that reason such beams may be taken as having a very long overhang. Slip in the central portion of a simple beam makes it act more and more like a bowstring girder as the slip progresses toward the ends, while a continuous beam has this tendency checked at the points of inflection. The same thing occurs in a flat slab. The graphs of the reinforcing rods given in the various tests of flat slabs supported on columns exhibit the same phenomenon of a knee as do simple beams, but usually at a much lower unit stress than 6 000 lb.

Apparently, the sharp change of direction of the graph at the so-called knee is a complex result dependent both on shrinkage and slip, which are related to each other in flat slabs somewhat otherwise than in beams, for this reason, if for no other, that such slabs have a depth not much more than half as great as beams of equal span, and the shrinkage grip of the concrete acts somewhat differently on crossed rods than on parallel rods. These, in some way, apparently, bring it about that the shrinkage stresses in the steel in flat slabs disappear at much lower steel stresses than in beams, and the slip is restricted to a small area at and around the column caps, so that the deflections due to this cause in the case of beams is very greatly reduced in the case of flat slabs.

The fact that the graph in beams is practically straight within the range of the working and test unit stresses from 16 000 to 30 000 lb., and that the neutral axis usually varies by much less than 10% of the depth on either side of the mid-depth, and, further, that the center of compression remains practically fixed in the beam for considerable variations of n, the ratio of the steel and concrete moduli, seem to destroy the cogency of Mr. Jonson's objections to the treatment of reinforced concrete on the basis of a suitably modified elastic theory.

It is slabs, however, rather than beams, which should claim our principal attention in this discussion. It appears that the most significant phenomenon exhibited by a panel supported at its edges by walls, or relatively stiff beams, is one which has been overlooked in the discussion thus far, a phenomenon which is exhibited by both slabs and plates in common. The experiments of Bach on plates supported in this manner showed that the failure of such plates was caused by their cracking along valley lines starting from each corner of the panel and almost exactly bisecting the corner angles, regardless of the square or oblong dimensions of the panel.

It is not difficult to imagine how this would occur, for, assume the plate to be supported, to begin with, on corner posts only, and after it has been deflected and has bent downward under the loading,

that it be gradually lifted by screw-jacks which raise stiff beams under the edges of the panel. In this way it is evident, at any time when the deflection of a side has been partly removed, that there is a sudden change of curvature which approaches discontinuity between the part of the slab that has been lifted on the beam and that part not yet lifted. Such a valley line, by reason of the comparatively sharp curvature across it, would be a line of great stress. Now, the analysis which has been proposed does not deal with this phenomenon at all, although it is the one thing of primary importance. The writer has attempted to treat this matter in the book previously referred to, and he knows of no other attempt at a theory of it. The attempted theory is shown to be in good accord with all available tests. Any theory which ignores, as does the present theory, this the most noteworthy experimental phenomenon of a thin slab with relatively stiff supports at the edges of the panels, cannot command our confidence, at least until it can be shown that it is in accord with experimental results such as seem to be totally lacking in this development. Mr. Eddy.

3.—Ratio of Lateral Distortion in Deformed Slabs and Plates.—The ratio of lateral distortion designated by K or by $\frac{1}{m}$ in the foregoing theory, which has been called Poisson's ratio, is in fact something very different from what is ordinarily designated by that phrase, and should no longer be called by that name. Poisson's ratio is the relative amount of the lateral compared with the longitudinal distortion, when a piece of material such as a flat plate or rod is elongated or shortened by direct stress. What occurs in the steel when a reinforced slab is subjected to bending is something quite different from the stretching of a flat plate, and is much more nearly analogous to the bending of a plate which has been stamped with a die into a pattern with alternate hills and valleys or dome-shaped convexities and saucer-shaped concavities, since that is the shape of the enclosed steel mat inside of a flat slab, a mat which, by reason of its embedment of interlocking concrete and its high comparative rigidity, acts not like a flat plate, but instead like a deformed continuous sheet of steel composed of many elevations and depressions. In order to form a correct idea of the deformation of such a sheet on bending, imagine a single saucer-shaped concave plate to be somewhat bent about a diametral line. One diameter of the saucer becomes less while that at right angles becomes greater. The radius of curvature of the normal section containing the smaller diameter thus becomes smaller while the radius of the other section becomes greater. The necessary connection and relation of these changes of curvature and the accompanying compressions and elonga-

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tions on the surfaces of the plate have nothing whatever to do with the molecular phenomenon which controls Poisson's ratio, but are due to the geometry of curved surfaces, although they might be modified somewhat by the magnitude of Poisson's ratio for the particular materials of which the curved plate is composed.

It is evident that, for a plate forming a segment taken from a hollow sphere of incompressible material, the distortion due to bending it would be such as to make the lateral and longitudinal distortions approximately equal. Thus K would be nearly equal to unity for such a substance, no matter how small Poisson's ratio might be; but, in such a plate, formed of some compressible material, K would be smaller. Its actual value would be a matter of experiment. It is this ratio of distortion of deformed plates that is contained in Equations (1), and not Poisson's ratio proper. The latter plays a very subordinate rôle in this theory, and the actual value of this ratio for the kind of material used has little or no relation to the value of K that must be used in these equations. It may be noted further that, in addition to the curvatures of the steel in the slab, the slab itself is likely, under its own weight and as an after effect of loading, to have some small deformations of a kind similar to those due to the curvatures of the steel mat.

The steel mat, in a flat slab on columns, however, does not consist entirely of convex and concave areas, for between these lie areas of twisted curvature which occupy approximately half of the total area of the slab. The mean value of K in Equations (1) depends in almost equal degree on these areas of saddle-shaped curvature and on those that are convex or concave. So far as the concurrent evidence of tests goes, the mean value of K for flat slabs supported on columns with the ordinary arrangement of steel is $K = 0.5$, although the value of Poisson's ratio for concrete alone certainly is not half as large as that. The manner in which K affects the deflections and resisting stresses of the steel is by the reduction factor, $(1 - K^2)$, and its influence, consequently, is much less than is often tacitly assumed by those who reason about it, as may be seen from the following tabulation:

$$K = 0.1 \quad 0.2 \quad 0.3 \quad 0.4 \quad 0.5$$
$$1 - K^2 = 0.99, \ 0.96, \ 0.91, \ 0.94, \ 0.75$$

from which it appears that, for values of K less than 0.2, its effect may be disregarded, but for $K = 0.5$, the effect modifies the values obtained by disregarding K by 25% at most.

So far as is known, this is the first time that it has been pointed out that the value, K, which must be used in equations of flat plates, is to be determined principally by considerations other than the molecular properties of the particular materials of which the plate is

composed, a fact which obviates the many criticisms which have been passed on any theory using any value larger than that found for the material of the plate. Those criticisms have assumed that, without question, the true value of Poisson's ratio for the materials composing the plate must of necessity be used in Equations (1), which is not the fact. Mr. Eddy.

The tests that have been made of flat slabs on columns have shown that, when the steel runs in belts over the columns, the stresses are almost entirely limited to the panels which carry the loads, and little effect is propagated from panel to panel. This has been thought by some to be due entirely to the stiffness of the supporting columns, as it undoubtedly is in part; but the foregoing discussion reveals an additional reason why the stresses in a panel are self-contained, much as it would be were it an arched or bow-string girder construction running to the columns. Such, however, is not the case to such a marked extent with two-way reinforcement in which rods cross the sides of the panels at mid-span and propagate moments more readily from panel to panel.

In the first of these constructions there are only comparatively small movements of the lines of inflection when a single tier of panels across a floor is subjected to a test load, and the action is not the same as it would be in a wide beam, because the action is more or less like attempting to collapse or flatten out a corrugated pipe instead of a plain pipe. It brings out the resistance of the reinforcement in a different and more effective manner.

It must be remembered that the actual statical moments of given loads in any panel cannot be altered by any process. The only question is how these moments are subdivided and resisted by the reinforcement. Other things being equal, that certainly occurs most economically when each panel is influenced least by adjacent panels, and at the same time each panel assists its surrounding panels as much as possible. In the case of a slab supported on relatively stiff beams or walls, each panel is largely independent of surrounding panels, and the lines of inflection lie relatively near the beams. The relative flexibility of the the slab is so large, and the deflection of the beams which support it is relatively so small, that the effect of their deflection is less important than seems to be imagined by Messrs. Janni and Jonson, and is negligible compared with the other inaccuracies of the method.

L. J. MENSCH,* M. AM. SOC. C. E. (by letter).—The author's assumption for the distribution of loading of the central strips of a rectangular slab is probably not far out of the way; yet his theory is misleading, because it is as little proper to assume that the average Mr. Mensch.

* Chicago, Ill.

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bending moment for such a slab should be figured as the moment of the central strips as it would be to assume that the eye-bar of a pin-connected bridge is strained only for a width equal to the diameter of the pin. During the last 12 years many investigators have divided rectangular slabs into a number of strips, thereby obtaining a clearer view of the behavior of the slabs. Danusso went further,* and investigated the influence of the diagonal strips which connect the center of the supporting girders and others parallel to them and nearer to the corners, and found that they greatly relieve the central strips. According to his theory, the average bending moment for a square slab freely supported on four sides is about $\frac{w\,l^2}{30}$, and, for one fixed on four sides, the average bending moment is about $\frac{w\,l^2}{70}$ in the center of the span and $\frac{w\,l^2}{34}$ at the supports. This seems to agree with nearly all recorded tests on such slabs.

Very instructive tests were made on two slabs of the "Palais de Costume" at the Paris World's Fair in 1902. One slab was about 22 ft. square, 6 in. thick, reinforced by $\frac{5}{8}$-in. round bars about $6\frac{1}{2}$ in. from center to center, and supported by light girders on very light columns. At a total load of about 500 lb. per sq. ft. the girders and columns were at the point of failure, and the girders had to be supported by temporary props and the test discontinued. The slab itself did not show any sign of very near failure, indicating that the negative bending moments produced great torsional stresses in the narrow girders and dangerous eccentric loadings in the light columns at a stage of loading which was probably only two-thirds of the ultimate carrying capacity of the slab. Better results were obtained with another slab, about 21 by 24.5 ft., supported on girders 18 in. wide and 24 in. deep, which girders were of considerably greater span than the slab proper. In this case, also, the girders failed first, which happened at a loading corresponding to an average bending moment in the slab of $\frac{w\,l^2}{38}$. The slab itself did not show any very near approach to failure.

We can cite also the very thorough tests made by Professor Bach, in 1914, on square and rectangular slabs simply supported on four sides. For comparison, Professor Bach tested also beams simply supported on both ends, having the same depth and the same percentage of reinforcement as the slabs. The diagonal strips (although the reinforcement ran only in two directions) were carrying a great part of the load on the slab, as was shown by the direction of the

* *Il Cemento*, 1912.

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cracks on the underside of the slab, and the average moment at the ultimate load was found to be about $\frac{w\,l^2}{28}$. The tests on rectangular slabs, with side ratios of 1:1½ and 1:2, and with the same percentage of reinforcement in both directions, gave the remarkable result that the ultimate load per square foot was from 77 to 85% of that obtained for a square slab the length of the sides of which was equal to the smaller side of the rectangular slabs. This can only be explained by the very pronounced action of the diagonal strips and by the fact that the limit of elasticity is first reached in the short direction, producing comparatively large deflections, thus allowing the longer span to take up the remainder of the load.

The writer heartily agrees with Mr. Jonson that the theory of reinforced concrete, as found in nearly all textbooks, ordinances, and in the reports of the Joint Committee, is rather crude, and should be entirely discarded. The theory is wrong because the plane-section theory and Hooke's law do not apply to reinforced concrete, even at ordinary loads—a fact proved by Professor Schule 14 years ago—and, as may be expected, is far from the truth at ultimate loads. Because we encounter very great difficulties in investigating the properties of any material near rupture, making it practically impossible to measure the flow or plasticity of the materials, the great mathematicians of about 100 years ago developed theories for the strength of materials based on stresses below the elastic limit, which, although they were a great improvement over the guess methods previously used, do not lead to constructions having the same factor of safety in every part. Following faithfully their lead, we were taught to allow stresses of about one-fourth of the ultimate strength of the materials, and have trusted ever since to good luck that our structures, designed according to these rules, have a factor of safety of about 4.

During the last 30 years a great number of testing laboratories have been erected in various parts of the world, and we now have more information about the strength of structures than ever before. Very slowly the fact has been brought out that our theories do not explain the ultimate strength of many members of constructions, and that it is advisable to use different working stresses for such members. This Society has recently helped to make most thorough tests on structural steel columns. No theory, according to the old standards, seems to fit these tests, but, if we take into consideration the properties of steel at the time of failure, especially the fact that the modulus of elasticity near rupture is considerably less than the initial value of 29 000 000 lb. per sq. in., on account of the flow of the material, and if we introduce the correct value of E in Euler's

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formula, we readily obtain a good agreement between theory and test. Of course, where $\frac{l}{r}$ is large, the ultimate unit stress is below the elastic limit, and in this case E equals 29 000 000, and for such long columns Euler's formula always gave satisfactory results. Similar discrepancies in old established theories were lately found also for rolled I-sections in bending, and there is no doubt that tees, angles, and other unsymmetrical sections will show still greater variations. If such is the case with a homogeneous material like steel, we may question in a still greater degree all reinforced concrete theories which are based on Hooke's law and the plane-section theory.

The writer has shown* that neither the plane-section theory nor Hooke's law is required to explain the results of tests on reinforced concrete girder and slab construction which failed without bond or diagonal tension failure.

Fig. 4 represents the ideal stress distribution in a section of a beam at the time of failure. Let f_c = the ultimate strength of concrete, in pounds per square inch, as shown in cylinder tests. Let f_s = the ultimate strength of reinforcing steel in reinforced concrete construction, which experience shows is about a mean between the elastic limit and ultimate strength, and all other notations as given by the Joint Committee; then, from the necessary condition that total compression equals tension, we have

$$\frac{2}{3}\, b\, k\, d\, f_c = p\, b\, d\, f_s$$

$$k = 1.5\, \frac{f_s}{f_c}\, p$$

$$M = p\, b\, d\, f_s \left(d - \frac{3}{8}\, k\, d\right) = p\, b\, d^2\, f_s \left(1 - \frac{3}{8}\, k\right)$$

or $$\frac{M}{b\, d^2} = p\, f_s \left(1 - \frac{3}{8}\, k\right)$$

or $$M = A_s\, f_s\, j\, d, \text{ when } j = \left(1 - \frac{3}{8}\, k\right).$$

The writer has shown (in the paper mentioned) that by assuming f_s = 45 000 lb. per sq. in. for mild steel, 72 000 lb. for high-carbon steel, and 110 000 lb. for commercial high-carbon drawn-steel wire, the ultimate moments, according to this formula, agree, within a small percentage, with the ultimate moments of nearly all published scientific tests on beams which he could find. Hence, the value of n is of no account in the investigation of the strength of a beam. In regard to its probable value at the ultimate load, it is known that the steel reinforcement in nearly all tests first shows signs of failure

* *Journal*, Am. Concrete Institute, December, 1914.

Mr. Mensch.

before the concrete is highly strained in compression, which means that the steel reaches and exceeds the elastic limit long before the concrete attains it in compression; when the steel exceeds the elastic limit, E drops very suddenly, and its probable value, just before rupture of a reinforced concrete beam, is less than 1 000 000 lb. per sq. in. The concrete in compression is not as plastic as steel, and it is probable that its modulus of elasticity near rupture is not less than from 300 000 to 500 000 lb. per sq. in., or the probable value of n at the ultimate load $= \frac{1\,000\,000}{500\,000} = 2$, and not—as Mr. Jonson states—30 or 40. Mr. Jonson's statement that "all calculations of reinforced concrete made on the assumption that the ratio, n or E divided by E_c, is a constant are unreliable * * *" shows that he is not familiar with the literature on reinforced concrete or statically indeterminate structures. Except for the influence of temperature

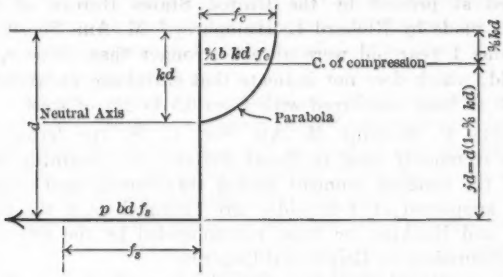

FIG. 4.

variation and shrinkage, the stresses in statically indeterminate structures never depend on the absolute values of $E\ I$, but on a function of the ratio of the value of $E\ I$ at any particular point, x, to a fixed value of $E\ I$ at any important point. Where $E\ I$ is constant, the moment of inertia has no influence whatsoever on the stresses caused by the indeterminate values, and, where I varies, the stresses depend generally on a function of the ratio of $\frac{d^3}{d_x^3}$ for rectangular sections. Assuming even that the moment of inertia is not proportionate to the 3d power of the depths of the section, the mistake appears only in a function of the ratio, and is of small influence; a mistake of 100% in the moment of inertia causes often a change in the stresses of only a few per cent.

Professor Morsch investigated this subject about 8 years ago and found a very fair agreement between theory and practice. More scientific tests were made by Professor Scheit and Mr. E. Probst on

Mr. Mensch. continuous girders on three to five supports. It is well known that, in a girder of constant moment of inertia, on three equidistant supports, the statically unknown moment over the central support is $\frac{Wl}{8}$. Their tests showed that the girder failed at the moment over the central support of $\frac{Wl}{10}$. The actual bending moment is smaller than that given by the elastic theory, which is easily explained by the fact that the elastic limit over the supports is reached long before the center of the spans is strained to the elastic limit, which has an effect of equalizing the moments over the supports and in the center of the span. These investigators obtained similar results for girders on four and five supports and for framed structures.

The influence of shrinkage of concrete on the strength of reinforced concrete beams is not solved yet, but is being thoroughly investigated at present by the United States Bureau of Standards. The tests made by Richard L. Humphrey,* M. Am. Soc. C. E., show that beams 1 year old were always stronger than those 4, 13, or 26 weeks old; which does not indicate that shrinkage unfavorably affects beams 13 ft. long reinforced with from 0.5 to 2% of steel.

Mr. Marsh. CHARLES F. MARSH,† M. AM. SOC. C. E. (by letter).—The coefficients commonly used in Great Britain, for obtaining the proportions of the bending moment acting transversely and longitudinally in slabs supported on four sides, are obtained from the formulas of Grashof and Rankine, or those recommended by the French Government Commission on Reinforced Concrete.

An inspection of the values given by these formulas will show that in both cases the transverse coefficient approaches unity as the ratio, $\frac{l_1}{l}$, increases, but never reaches this value; and the longitudinal coefficient approaches but never reaches zero as the ratio, $\frac{l_1}{l}$, increases, and, from the nature of the case, these would appear rational and obvious results.

By the use of Mr. Janni's formulas, the transverse coefficient is unity and the longitudinal coefficient is zero when $\frac{l_1}{l} = \frac{5}{3}$, and their use for ratios of $\frac{l_1}{l}$ greater than $\frac{5}{3}$ gives a transverse coefficient greater than unity and a minus quantity for the longitudinal coefficient. Such results do not impress one as being in accordance with the facts.

* *Technologic Paper No. 2*, Bureau of Standards.

† London, England.

Mr. Janni obtains his formulas by considering the slab as divided into systems of beams parallel to the short and long sides of the slab and equating their deflections at the center; the formulas of Grashof and Rankine are obtained by the same method. Mr. Marsh.

The introduction of the effect of Poisson's coefficient is doubtless essential, from an academic point of view, but the practical utility and accuracy of the results are of doubtful value, especially when, as a preliminary, the important assumption is made that the moment at P is reduced to two-thirds of its theoretic value by reason of the doubtfulness as to the efficiency of the fixation.

With respect to Mr. Jonson's discussion, it might be well to point out that the generally accepted methods of treatment, working stresses, and coefficients were recommended by the various authorities after full consideration and knowledge of the general effect of the various influences other than those actually provided for in the formulas, and there appears to be no reason to doubt that structures designed by the use of such formulas and the stresses and coefficients recommended will, if properly constructed, be amply sufficient to serve the purposes for which they are designed.

The initial strains due to the shrinkage of the concrete when setting do without doubt have an effect on the structure, but other conditions also affect the resistance, such as loading and unloading, temperature and humidity, the greater facility of the reinforcements than of the concrete in recovering after straining, etc.

In any case, it is very doubtful whether Mr. Jonson's conclusions with respect to the effect of the initial strains due to shrinkage are correct, and it is more than probable that these strains would add to the resistance of a reinforced concrete member.

Shrinkage of the concrete induces compressive stress in the reinforcements and tensile stress in the concrete and, in the case of a member subjected to flexure, if the concrete does not crack on the compression side of the neutral axis, the resistance of the member is increased; but if the concrete cracks, as the initial stresses are induced before the cracking occurs and the crack must close up before the concrete can be stressed by the loading, the initial strain of the reinforcement must be removed, leaving matters as they were before the shrinkage.

In the case of a member under direct compression, the initial compressive stress on the steel added to the compressive stress due to loading will never approach the permissible working stress on the steel.

From the results of published experiments it would appear that the value of Em for 1:2:4 concrete is not likely to have a range greater than from $-$ 3 000 when the setting takes place in water to $+$ 6 000 when the setting takes place in air.

Mr. Marsh.

With respect to the value of the coefficient, n, it is well known and recognized that its value may vary between wide limits, and that the value used in calculations must necessarily be in a great measure hypothetical.

Many experiments have been made with the object of ascertaining a proper value for this coefficient, and the results have varied considerably, which is hardly surprising, considering the nature of the material under test.

Mr. Jonson concludes, from certain recently conducted experiments, that the modulus of elasticity of concrete decreases with age, whereas similar experiments in the past have indicated an increase. From a careful study of experiments bearing on the value of this modulus conducted up to about 1908, it was concluded that a fair average value was about 1 740 000 under a stress of 600 lb. per sq. in., giving n a value of about $17\frac{1}{2}$, but that the tendency of the concrete to take a permanent set, while the steel was truly elastic, would have an effect equivalent to a decrease in the value of n, and, consequently, it was concluded that the adoption of a value of 15 for n was a reasonable assumption. It was also found that the results of actual experiments on reinforced concrete beams justified this value.

The treatment of continuous construction as consisting of a series of cantilevers supporting beams at their ends is a method which is frequently adopted, but its accuracy is questionable, especially where the loading may vary and the spans are unequal.

For the ordinary floor design, it would appear that approximate coefficients for the bending moments, such as those given in the London County Council Regulations, might be used with perfect safety and with a great simplification of the calculations.

Mr. McMillan.

FRANKLIN R. MCMILLAN,* ESQ. (by letter).—In the following remarks attention will be given to the questions raised by Mr. Ernst F. Jonson, in his discussion, rather than to the method of calculation proposed by the author; for these questions, which concern the fundamental calculations of reinforced concrete beams, must be settled before we are justified in passing to the refined methods suggested by Mr. Janni.

During the past 3 years, the writer has been conducting, at the Experimental Engineering Laboratory of the University of Minnesota, a series of investigations concerning some of the physical properties of concrete which Mr. Jonson has mentioned as affecting the theory of flexure of reinforced concrete. Although these investigations are still far from complete, considerable data have been accumulated which relate to the questions at issue.

As the value of such calculations as made by Mr. Jonson depends on the correctness of the assumptions, it will be well to present a sum-

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Mr. McMillan.

mary of the more important facts and conclusions from these investigations before taking up the discussion.

These results relate principally to the shrinkage and the non-elastic deformation or time yield in reinforced concrete beams, slabs, and columns.* The curves of Figs. 5, 6, and 7 show the results of shrinkage measurements on a number of specimens designed to bring out the effect of several variables. In regard to Fig. 5 little need be said in explanation, as the legends show quite clearly the variations. These results were obtained from 4 by 5 by 24-in. specimens, cast under conditions as nearly identical as possible, and cured side by side, 2 weeks under wet burlap and then open to the ordinary air of the laboratory. Initial readings were taken when the beams were one or two days old, hence the values given represent practically the entire shrinkage. From these curves it will be seen that the total shrinkage is from 0.055 to 0.065%, and that little difference results from varying the proportions, quantities of water, or density, within the limits of ordinary practice. It will also be observed that in nearly every case the shrinkage amounts to 0.05% within the first 60 days.

On Fig. 6 are shown the results of shrinkage measurements extending over periods of from 2 to 3 years, in which somewhat higher values are found than in the specimens of Fig. 5. It will be noted that there is a large difference in the total shrinkage shown by the two slabs in the upper diagram, and that the greater value is shown for the slab on which observations were not commenced until it was 68 days old. As this slab had been covered with wet sand until within 3 days of the zero measurements, there may have been some swelling, which, if subtracted from the values shown, would give the shrinkage from the original state. In some other tests, not included here, a swelling of nearly 0.02% has been observed in 80 days under wet burlap. The concrete of the 6 by 8-ft. slab is considerably more porous than that of the 10 by 10-ft., which may in a measure account for the higher shrinkage, as well as for the quicker response to the change in the humidity of the air, as seen from the undulations of the shrinkage curves. It should be noted that the periods shown on this or any of the other figures do not represent coincident calendar periods for the different specimens; they were plotted to the same set of co-ordinates for simplicity. When plotted on a true calendar scale it is found that the undulations are coincident, showing the seasonal changes in the humidity.

In the lower diagram of Fig. 6 rather high values of total shrinkage are shown; the effect of early wetting on the total shrinkage can also

* For a more extended discussion of these tests, see *Bulletin No. 3, Engineering Series*, University of Minnesota, "Shrinkage and Time Effects in Reinforced Concrete". Also the paper, "Time Tests of Concrete," by F. R. McMillan, *Journal*, Engineers' Club of St. Louis, Vol. 1, No. 4.

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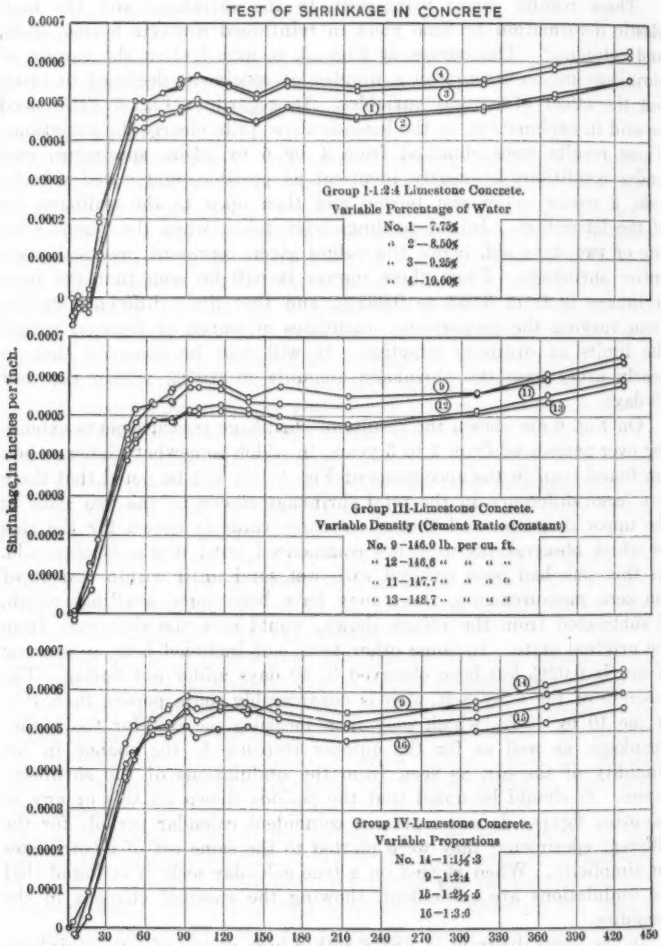

FIG. 5.

be seen. These two beams were identical, except for the methods of curing, as noted on the diagram. That the effect of the early wetting on Beam *B*—or the subsequent immersion—is only to retard the shrinkage, and does not affect the final total amount, can be seen from these curves. This is also indicated by the results from the

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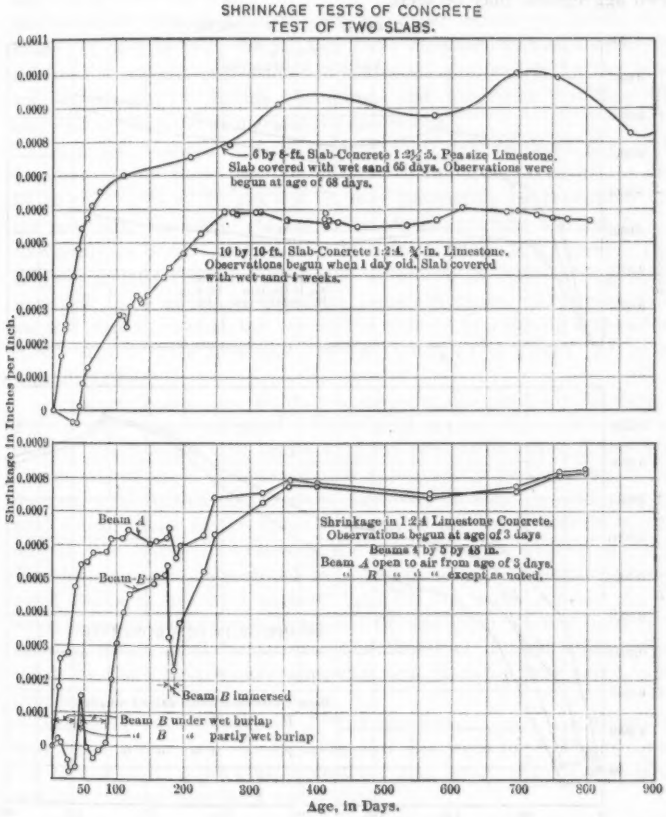

FIG. 6.

6 by 8-ft. slab previously referred to, which show a total shrinkage of nearly 0.1 of 1% beginning after curing 57 days under moist sand.

In the upper curves of Fig. 7 are given the results of measurements on 4 by 5 by 24-in. beams, made from 1:2:4 concrete of three different aggregates, and cured under conditions as nearly identical as possible.

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These results are presented as further evidence of what total shrinkage may be expected, rather than for the comparison of the different aggregates, for with these few tests definite conclusions cannot be drawn. In the lower curves the effect of early wetting on both the limestone and sandstone concrete, as well as the comparison of the two aggregates, may be seen.

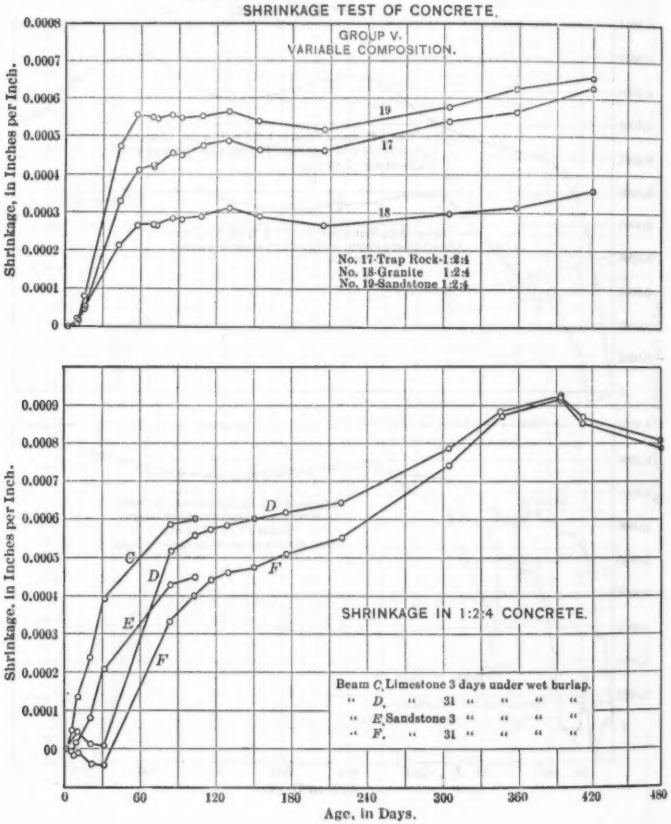

FIG. 7.

That other variables than those controlled in the several series of these tests affect the amount of shrinkage is evident from the varying amounts shown by specimens of nominally the same mixture; for example, the 1:2:4 limestone concrete, shows the following values

for the different tests: in Fig. 5, from 0.055 to 0.065%; in Fig. 6, from 0.06% for the 10 by 10-ft. slab to 0.08% for Beams *A* and *B*; and, in Fig. 7, 0.09% for Beam *D*. This is further illustrated by the values of 0.065 and 0.092% shown by the upper and lower curves for the 1:2:4 sandstone concrete of Fig. 7. These differences are considerably greater than found for those tests shown on Fig. 5 in which the variables were regulated. Among other causes to which these differences may be due, the following are suggested as probably having considerable weight: variations in materials assumed to be the same, and differences in mixing, tamping, and otherwise handling the materials at the time of making the specimens. Mr. McMillan.

From the foregoing it is evident that definite laws cannot be formulated from these tests; however, two important conclusions may be drawn: First, as to amount, it may reasonably be concluded that the minimum shrinkage to be expected in concrete allowed to dry thoroughly is 0.05%, and that values from 20 to 50% greater than this will usually result; second, the difference in early curing conditions has little effect on the final total shrinkage. Concrete cured wet will swell somewhat, but, when it is exposed to a drying atmosphere, it will shrink fully as rapidly and as much as if placed in the air when first cast.

In regard to the time yield or non-elastic deformation, the data on Figs. 8 and 9 are presented. On Fig. 8 appear the results of tests on four small beams kept under load for periods of 200 and 500 days. The details of these tests are shown quite clearly by the notes on the diagrams; in further explanation, however, it may be said that the total deformations at the upper surfaces of the beams were measured over an 8-in. gauge line crossing the point of application of the load. The shrinkage was determined from measurements on identical beams cured similarly but held without load for the same period. As the stress increases rather rapidly toward the center, in such a short span, the total measured deformations—and therefore the net deformations shown by these curves—are somewhat less than the true value, probably from 5 to 10 per cent.

Beams 11 and 13 were of 1:2:4 concrete, of pea size limestone, kept wet for one day. Beams 118 and 121 were of 1:2:4 concrete, the former with ¼ to ¾-in. limestone, the latter with Kettle River sandstone; these were kept wet 3 days.

An inspection of the curves of Fig. 8 shows that the moduli of total deformation reduce considerably in the periods for which these measurements were made. Table 2 gives the ratio of the deformation shown by these curves at several different ages to the initial deformation.

In Table 2 the remarkable regularity in the values for the three limestone beams, 11, 13, and 118, should be noted. Although the

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initial moduli may differ, the rate of the progressive movement is about the same. No determination of the elastic modulus was made on the concrete of these beams at the time of loading, but calculation from the bending moment indicates values ranging from 1 600 000 to 2 200 000 for the limestone, and 3 000 000 for the sandstone. In these calculations the conditions of a homogeneous beam, 4 by 5 in. in cross-section, were assumed, as it is known that no cracks developed at first loading. On the basis of these values, we may expect at 200 days a value of n from 26 for the sandstone to 50 for the limestone, and from 50 to 60 for the limestone at 500 days.

TABLE 2.

Beam.	Initial deformation, in inches per inch.	Ratio of Net Deformation to Initial Deformation.			
		30 days.	90 days.	200 days.	530 days.
11	0.000385	2.49	2.66	3.50	4.09
13	0.000302	2.48	2.78	3.28	3.74
118	0.000246	2.40	2.84	3.29	....
121	0.000180	1.57	2.08	2.68	....

By net deformation is meant total deformation minus the shrinkage. It includes the deformation due to load and time yield.

TABLE 3.

Specimen.	Coarse aggregate.	Nominal proportions.	Time kept wet.	Age at Application of the Various Loads.	
				Dead load.	Other loads.
Beam *A*....	Limestone, pea size.	1 : 2 : 4	1 day	.....	Dead load and 860 lb. at center applied at age of 33 days.
Beam *B*. ..	Limestone, ¼ to ¾ in.	1 : 2 : 4	(not wet)	41 days	Loaded at one-third points. Total of 500 lb. at 80 days. After several changes, on and off, total of 1 500 lb. at 88 days
Slab *C*.....	Limestone, ¼ to ¾ in.	1 : 2 : 4	31 days	135 days	Live load 50 lb. per sq. ft. at 309 days. Increased to 100 lb. at 410 days.
Slab *D*.....	Limestone, pea size.	1 : 2½ : 4	57 days	59 days	Live load 50 lb. per sq. ft. at 68 days. Increased to 100 lb at 71 days. Live load removed at 82 days, and re-applied at 94 days.

On Fig. 9 are shown the net deformation curves for two more beams and two slabs. These curves differ from those of Fig. 8 in that the load was not applied at one time. For example, in the curve for the 10 by 10-ft. slab the deformation shown at the first day was due to

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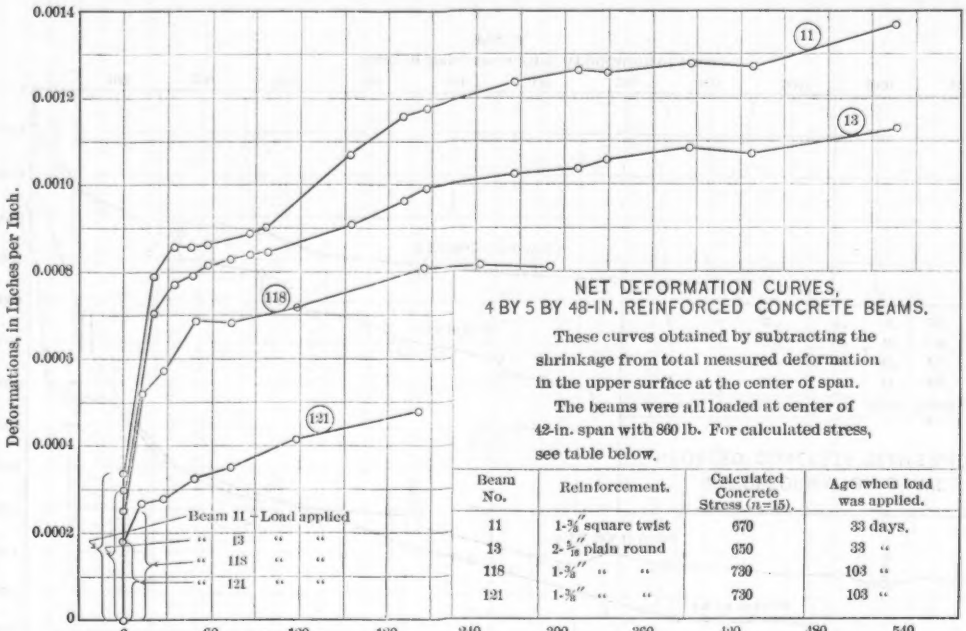

NET DEFORMATION CURVES,
4 BY 5 BY 48-IN. REINFORCED CONCRETE BEAMS.

These curves obtained by subtracting the shrinkage from total measured deformation in the upper surface at the center of span.

The beams were all loaded at center of 42-in. span with 860 lb. For calculated stress, see table below.

Beam No.	Reinforcement.	Calculated Concrete Stress (n=15).	Age when load was applied.
11	1-$\frac{3}{8}$″ square twist	670	33 days.
13	2-$\frac{5}{16}$″ plain round	650	33 "
118	1-$\frac{3}{4}$″ " "	730	103 "
121	1-$\frac{3}{8}$″ " "	730	103 "

FIG. 8.

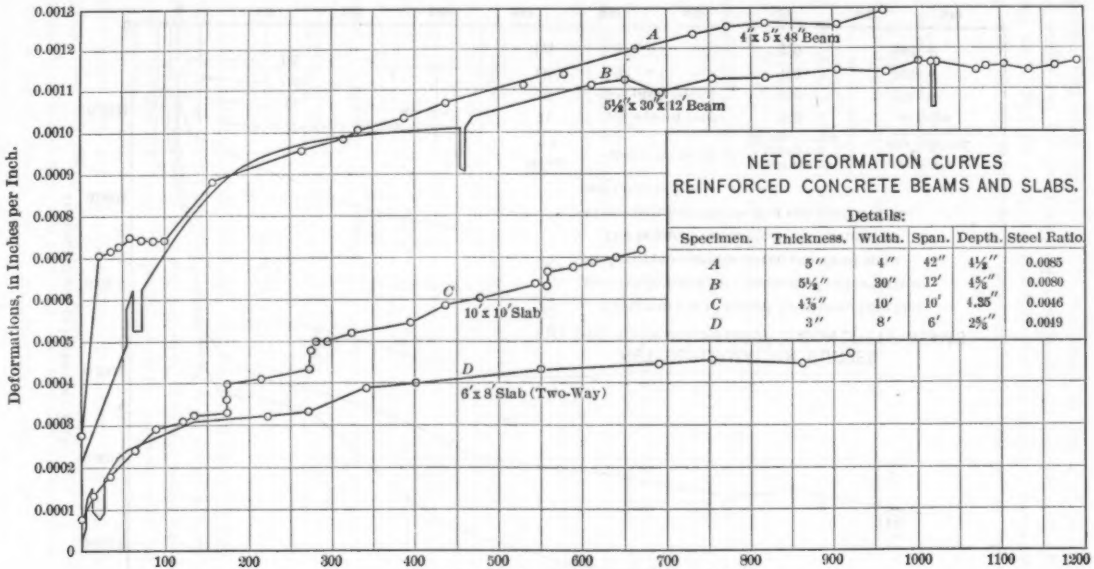

FIG. 9.

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the dead load only (55 lb. per sq. ft.), and no further load was applied until the 174th day, when a live load of 50 lb. per sq. ft. was added, with a similar increase 100 days later. The increase in deformation at 560 days was the result of an overload applied for the purpose of breaking the concrete in tension, which, up to this time had shown no signs of cracking. Table 3 gives the mixture, age, and loading of these beams and slabs, and Table 4 the ratios, at different ages, of net deformation to load deformation. In the calculation of these ratios, the net deformation at the given age, as shown by the curve on Fig. 9, is divided by the sum of the load deformations for all previous load increments. Thus, for the 250th day on the 10 by 10-ft. slab, the ratio is 3.2, obtained by dividing 0.000428, the net deformation from the curve, by (0.000076 + 0.000058), the dead-load deformation plus the live-load deformation on the 174th day.

TABLE 4.

Specimen.	Load Deformations.					Ratio: Net Deformation to Load Deformation at:				
	Dead load.	Live load.	Time, in days.	Live load.	Time, in days.	50 days.	100 days.	200 days.	650 days.	950 days.
Beam *A*....	.....	0.000280	0	.....	...	2.57	2.68	3.39	4.25	4.60
Beam *B*....	0.000200	0.000035	46	0.000075	54	....	2.32	3.11	3.63	3.71
Slab *C*.....	0.000076	0.000058	174	0.000043	275	2.90	....	3.20	4.00	....
Slab *D*....	.....	0.000042	0	0.000065	3	2.24	2.66	3.08	4.11	4.39

Time refers to interval from first application of load.

In the ratios of net to load deformation in Table 4, practically the same values are found as for the corresponding periods in Table 2. The initial moduli for three of these specimens were determined, with the following results:

Beam *B*..... 3 460 000 at 27 days.
Slab *C*..... 3 300 000 at 135 days.
Slab *D*..... 2 500 000 at 72 days.

These give values of n ranging from 9 to 12 at the time of application of the loads, from 20 to 30 at 3 months, and from 33 to 53 after 2 years.

The conclusions to be drawn from these tests of shrinkage and time yield, which apply to the question raised by Mr. Jonson, may be summed up as follows: First, in concrete allowed to dry thoroughly a shrinkage of from 0.06 to 0.07% may ordinarily be expected, with a minimum value of 0.05%; second, the modulus of total deformation will be from one-half to one-third of the initial modulus in a few months, and about one-fourth or less after 2 years; that is, on the

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usual basis of an initial value of n ranging from 9 to 15, values from 20 to 30 may be expected in a few months and from 35 to 60 at 2 years.

Now, to apply the values just quoted for shrinkage and time yield to the calculation of beams in the manner followed by Mr. Jonson, the upper diagram of Fig. 10 has been prepared. The formulas of Mr. Jonson from which this diagram was constructed assume the condition that the beam is not free to shrink, and therefore the concrete cracks, a condition that may be possible, though probably not the usual one.

The diagram referred to assumes a beam with a steel ratio of 0.0077 and loaded so that $\frac{M}{b\,d^2} = 107.4$, the value corresponding to unit stresses of 16 000 and 650 for $n = 15$. The true stresses in such a beam for various values of n can be read directly from this diagram. The two sets of curves show the extreme cases of shrinkage, the value, 0.01% ($Em = 3\,000$), being the minimum suggested by Mr. Jonson, and the other, 0.07% ($Em = 21\,000$), being the probable ordinary maximum, as shown by the experiments quoted. From these curves it is seen that, with a shrinkage of 0.07%, a beam designed by the usual assumption would have a concrete stress anywhere from 800 for $n = 20$ to 580 for $n = 50$, and a steel stress of from 15 600 to 16 300, within the same limits. For a shrinkage of 0.05% the values would be a little less for the concrete and but slightly more for the steel.

The quantity of steel to use for any unit stress desired can best be determined from the lower diagram of Fig. 10, which has been prepared for a fixed unit stress and several ratios of steel. To illustrate: With 650 as the desired limiting concrete stress and $n = 40$ assumed to represent the probable condition of total deformation, by interpolation from this diagram it is seen that a steel ratio of 0.0085 would be required, with the resulting value of $\frac{M}{b\,d^2}$ at 117. This is 10% more steel than required for the usual assumptions, and gives 9% greater resisting moment. For $n = 30$, the same concrete stress would require only 0.7% of steel, and would give a value of 100 for $\frac{M}{b\,d^2}$; this is 9% less steel and 7% less moment than for the beam designed on the usual assumptions.

Fig. 11 is similar to the lower diagram of Fig. 10, using the value of 3 000 for Em. From this diagram it is seen that, for a concrete stress of 650, 1.2% of steel would be required for $n = 40$, and 1.03% for $n = 30$. The corresponding values of $\frac{M}{b\,d^2}$ are 153 and 138, respectively. When compared with a beam designed by the usual

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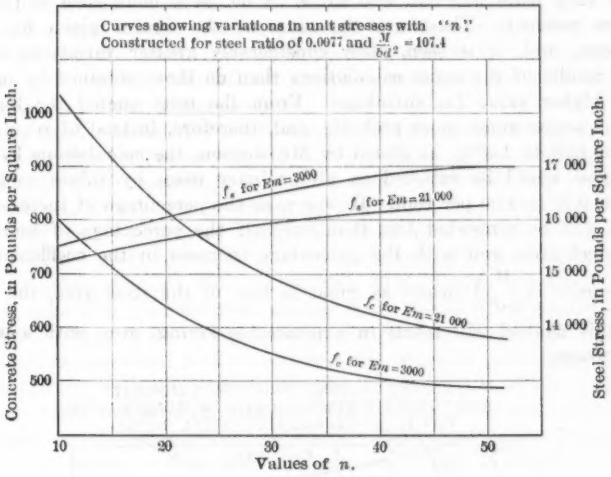

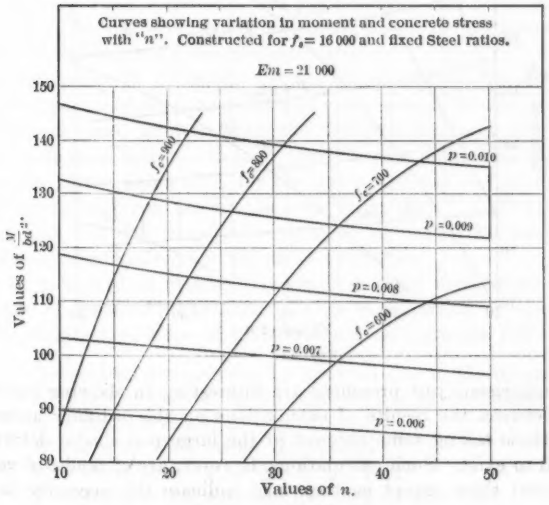

FIG. 10.

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assumptions, the values for $n = 40$ give a beam of 56% more steel and 42% more moment, and for $n = 30$, 34% more steel and 28% more moment. These are the same as the figures given by Mr. Jonson, and, it is seen, show considerably greater variations from the results of the usual calculations than do those obtained by using the higher value for shrinkage. From the tests quoted the higher value seems much more probable, and, therefore, instead of requiring from 1.03 to 1.20%, as stated by Mr. Jonson, the calculations by his method would be satisfied in all ordinary cases by values ranging from 0.75 to 1.00 per cent. However, as the percentage of increase in cost will be somewhat less than one-half the percentage of increase in steel area, and with the percentage increase in the coefficient of resistance $\left(\frac{M}{b\,d^2}\right)$ nearly as great as that of the steel area, the use of this method will result in a noticeable saving, even with a large shrinkage.

CURVES SHOWING VARIATION IN MOMENT AND CONCRETE STRENGTH WITH "n" CONSTRUCTED FOR $f_s = 16\,000$ AND FIXED STEEL RATIOS.

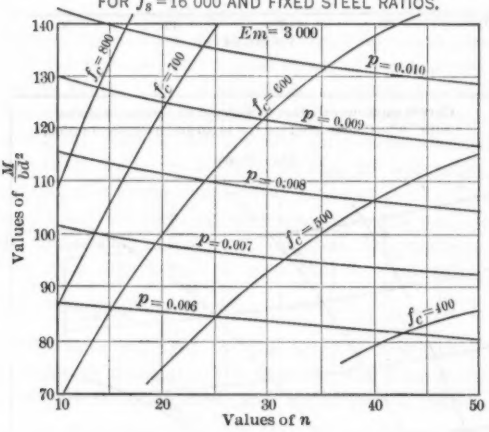

FIG. 11.

The comparisons just presented are interesting in showing possible variation between the results of calculations on the ordinary assumptions and those taking some account of the large non-elastic deformations found to exist. Such calculations, however, are of doubtful value at the present time, except as they may indicate the necessity for a thorough study and revision of the theory of reinforced concrete beams, for they are based on assumptions varying enough from the real con-

Mr. McMillan.

ditions to involve greater errors, perhaps, than those they seem to have pointed out. For example, the assumption that the beam is anchored so that the steel does not change in length from the shrinkage is probably not often realized in practice, and, if so, the concrete will probably crack, if at all, at irregular intervals rather than in the manner assumed in the analysis.

Until further knowledge is obtained of the inter-action of the materials under the various conditions that are likely to be encountered in the completed structure, it does not seem desirable that any change from the present practice in design be attempted; for, as regards the stresses in simply supported beams, as Mr. Jonson has pointed out, the time yielding must result in a readjustment of abnormally high initial concrete stresses. When a revision of the theory is attempted, it must take into consideration, among other things, such conditions as the following: A beam simply supported and free to shrink, with an initial compression in the steel and the concrete uncracked; the same condition with the concrete cracked at rather wide and irregular intervals from the bending moments applied, also with the concrete thoroughly cracked throughout its length. Restrained beams, because of bending cracks over or near the supports, divide themselves into sections which may fulfill any or all of the foregoing conditions. Although all these conditions are neglected in the common theory, they cannot be neglected when an attempt is made to introduce the non-elastic deformations into the calculations.

In the deflection of beams the shrinkage and time yield play an important part. Deflections measured after several months are several hundred per cent. greater than the calculations from the common theory of flexure would indicate. Mr. Jonson's method of calculating deflections, if the value of f_s for the uncracked sections is used, gives results probably much nearer the true value than his moment calculations give the true stresses, because of the small influence of the occasional tension crack on the total deflection.

It is in regard to column stresses that these non-elastic deformations are of greatest importance. Using the notation of Mr. Jonson, and to re-state here some of his equations, we have the following:

f_r = initial compression in the steel due to shrinkage;
f_t = initial tension in the concrete due to shrinkage;
m = unit shrinkage;
E and n in their usual significance;
f_y = stress in the steel due to the load;
f_x = stress in the concrete due to the load;
f_s = combined steel stress = $f_r + f_y$.
f_c = combined concrete stress = $f_x - f_t$.

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$$f_r = \frac{E\,m}{p\,n+1}, \text{ and } f_s = f_y + \frac{E\,m}{p\,n+1} \quad \ldots\ldots\ldots\ldots (1)$$

$$f_t = \frac{E\,m\,p}{p\,n+1}, \text{ and } f_c = f_x - \frac{E\,m\,p}{p\,n+1} \quad \ldots\ldots\ldots\ldots (2)$$

Now, the load applied to the column,

$$P = f_y\,p\,A + f_x(A - p\,A) \quad \ldots\ldots\ldots\ldots (3)$$

in which

$$f_y = f_x\,n \quad \ldots\ldots\ldots\ldots (4)$$

or

$$P = f_x\,A[1 + (n-1)\,p] \quad \ldots\ldots\ldots\ldots (5)$$

From these formulas can be obtained the actual stresses in columns designed by some of the accepted formulas of the day. For illustration, take two columns using 2% and 4% of vertical steel and 1½% of spirals. Calculated by the Joint Committee recommendations, the values of $\frac{P}{A}$ for these two columns would be 832 and 1 014, respectively. If calculated by the formula and unit stresses permitted in the Minneapolis Building Code:

$$P = 800\,Ac + 10\,000\,As + 2.4 \times 16\,000\,As'$$

the values of $\frac{P}{A}$ are 1 560 and 1 744, respectively.

Applying these formulas to these four columns, we get the actual unit stresses, f_c and f_s, shown in Table 5. These are worked out for two sets of values of Em and n, representing conditions that may frequently be expected.

TABLE 5.

$\frac{P}{A}$	p	$Em = 15\,000$, $n = 40$.				$Em = 21\,000$, $n = 50$.			
		f_x	f_y	f_c	f_s	f_x	f_y	f_c	f_s
832	0.02	467	18 680	300	27 000	420	21 000	210	31 500
1 014	0.04	396	15 840	166	21 600	343	17 150	63	24 150
1 560	0.02	876	35 040	710	43 370	788	39 400	578	49 900
1 744	0.04	682	27 280	450	33 040	599	29 500	310	36 500

From Table 5 it is seen that enormously high steel stresses may be expected in columns as conservatively designed as by the requirements of the Joint Committee, and that in columns in which large allowance is made for the spirals, stresses well above 30 000 lb. cannot be avoided. It should be noted that the shrinkage stress alone in these four columns varies from 6 000 to 10 500 lb. per sq. in. In support of the reasonableness of these calculations, the data on Fig. 12,

which show the results of measurements on columns of two buildings in service, are presented. Mr. McMillan.

In the first building the columns were 5 months old, and were carrying four floors and the roof when the observations were begun. Only a small quantity of dead load, such as partitions, interior finish, and equipment, has been added since. This is a university building, and, like others of its class, receives a very small proportion of the live load for which it was designed. The results of these observations are given in the curves marked Column 1 and Column 2. These

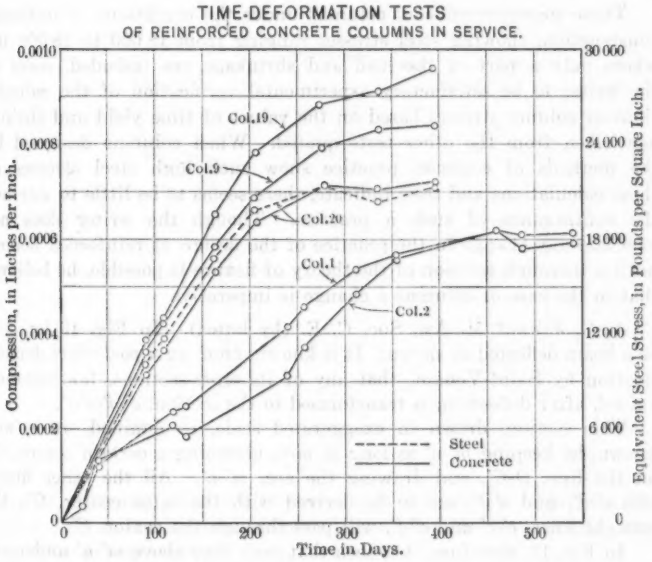

FIG. 12.

measurements were made at the surface of the column, but it is believed that measurements taken directly on the vertical rods would have shown the same results. The steel stress represented by these deformations is 18 000 lb. per sq. in., which is very noteworthy when it is considered that practically no dead or live load is included, and that at least some of the shrinkage must have taken place during the 5 months before the gauge lines were established.

The observations on the second building are given in the curves for Columns 9, 19, and 20. These were begun when the columns were about 2 months old, and were carrying only a portion of the full dead

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load; the results, therefore, probably show a considerable portion of the shrinkage, as well as some of the dead-load stress. Like the previous building, very little live load comes to these columns, although it is provided for in the design. For Column 20, observations on both the concrete at the surface and on the vertical steel are shown; these, it will be noted, are in practical agreement. The steel stresses from the measurements on these three columns are from 21 000 to 28 000 lb. per sq. in., with the indication that there may be still some increase. These are basement columns, and as yet they have not been thoroughly dried out.

These measurements on columns under the conditions of ordinary construction, showing steel stresses ranging from 18 000 to 28 000 lb., where only a part of the load and shrinkage are included, seem to the writer to be satisfactory experimental verification of the calculations of column stresses based on the values of time yield and shrinkage taken from the other tests quoted. When columns designed by the methods of common practice show such high steel stresses as these calculations and tests indicate, there seems to be little to warrant the continuance of such a practice. Though the writer does not advocate any change in the practice of the design of reinforced beams, until a thorough revision of the theory of flexure is possible, he believes that in the case of columns a change is imperative.

Mr. Janni.

A. C. JANNI,* M. AM. SOC. C. E. (by letter).—In Fig. 13 let *A-B* be a beam deflected as shown. It is known, from an *a posteriori* demonstration by Saint-Venant, that any of its cross-sections, for instance, *c-d-e-f*, after deflection, is transformed to the section, *c'-d'-e'-f'*.

This section, drawn in exaggerated scale, is obtained, as is well known, by keeping *m'-n'* as long as *m-n*, assuming a certain center, *C'*, on the line, *O-C'*, and drawing the arc, *m'-n'*. All the other fibers, like *c'-d'*, and *e'-f'*, are to be derived with the same center, *C'*; the straight lines, *e'-c'* and *d'-f'*, will pass through the center, *C'*.

In Fig. 13, therefore, it is seen that each fiber above *m'-n'* undergoes a shortening which increases with its distance from *m'-n'*. The reverse—that is to say, the stretching of the fibers—happens for all fibers below *m'-n'*.

Poisson's coefficient, $\frac{1}{m}$, is simply the numerical translation of the physical phenomenon just described, and everybody understands that, in order to take it into account in an exact formula, it cannot be represented graphically very well, hence, tests have been made and have brought out various values for $\frac{1}{m}$, and these are not so far from $\frac{1}{4}$,

* New York City.

its theoretical value, as some seem to think. The tests mentioned, of course, were made by reliable scientists. Mr. Janni.

The general formula for the exact computation of stresses in beam deflection contains, in fact, the coefficient, $\frac{1}{m}$, as can be seen:

$$k = \text{Max.}\ \frac{M}{I}\left(\frac{m-1}{2\,m}\,h + \frac{m+1}{2\,m}\sqrt{h^2 + 4\,S^2}\right) \ldots\ldots(24)$$

where, M, I, m, and h have the same meaning as given in the paper, and S is the shear.

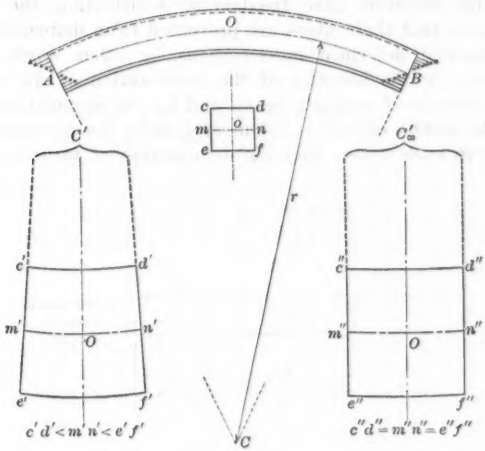

FIG. 13.

For every-day purposes, however, it is sufficient to verify the foregoing only at the top and bottom fibers and at the neutral axis, where it becomes, respectively:

$$k_1 = f_1 = \frac{M}{I}\,h, \text{ and } k_2 = f_2 = \frac{m+1}{m}\,S \ldots\ldots\ldots(25)$$

This shows that f_1, which we usually look for, is not a maximum, by any means, and that the indiscriminate use of this formula may lead to serious errors.

Fig. 13 shows, also, that, except for the neutral plane fibers, all the others of the cross-section are affected by Poisson's coefficient, and, therefore, the deflection of the beam is affected accordingly, although, for practical purposes, its influence may be ignored.

Now, if this same beam, A-B, is one of the ideal middle beams of a flat slab, its cross-section, c-d-e-f, on account of the continuity

Mr. Janni.

of the system of beams, cannot be deformed as the section, c'-d'-e'-f', was.

Then, we will have, as in Fig. 13, m''-$n'' = m'$-$n' = m$-n, and c''-$d'' = e''$-$f'' = m''$-n''; that is, the center, C', is at infinity and becomes C'_∞, and the fibers, c''-d'', e''-f'', will be parallel to m''-n'' as before, having, furthermore, a less pronounced curvature, which is the curvature of the ideal beam of the other system, crossing the first beam at that point.

We have before us, therefore, the shapes of the same cross-section of a deflected beam, the first by the hypothesis that the fibers of the beam during deflection have freedom of deformation, the second by the hypothesis that those fibers are prevented from deforming.

The enlarged deformed cross-sections, in other words, represent the difference in the behavior of the cross-section of the same beam by the hypothesis of a single beam, and by the assumption that this beam is the middle one of a system of parallel beams connected with each other in such a way that the deformation of its cross-section is prevented.

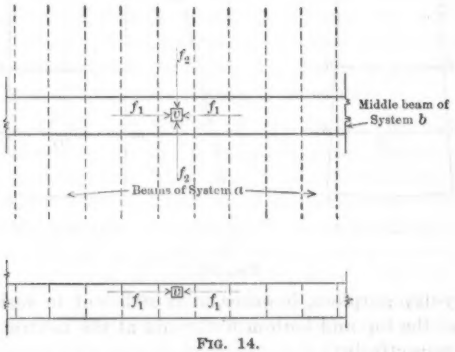

FIG. 14.

The absence of this phenomenon of deformation in this second case is essential to the behavior of the material in flat slabs, and, in order to take it into account in this investigation, it must be carried numerically in any formula the purpose of which is the finding of stresses in those ideal beams.

Accordingly, imagine that the ideal middle beam of System b is detached from all the beams of System a, Fig. 14, and is represented by v, an elementary cubical element of this beam.

Four forces will act on the vertical faces of this element. Two of these, f_1, are due to the deflection of the beam itself, and the other two, f_2, are due to the deflection of the ideal beam of System a, which, at that point, crosses the beam under consideration.

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Let e_1 and e_2 be the corresponding elastic deformations of the element, v, in these two directions; then,

$$e_1 = \frac{1}{E}\left(f_1 - \frac{1}{m} f_2\right) \ldots\ldots\ldots\ldots\ldots\ldots (26)$$

and,

$$e_2 = \frac{1}{E}\left(f_2 - \frac{1}{m} f_1\right) \ldots\ldots\ldots\ldots\ldots\ldots (27)$$

The deformation, e_2, must be zero, for the reason stated above.

We have, therefore,

$$f_2 = \frac{1}{m} f_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (28)$$

Hence,

$$e_1 = \frac{m^2 - 1}{m^2} \times \frac{f_1}{E} \ldots\ldots\ldots\ldots\ldots\ldots (29)$$

Now, if we consider two parallel contiguous sections of the beam, S_1 and S_2, as shown in Fig. 15, and a fiber, f, having a cross-section, dA, and a length, dx, the derivative of its work of deformation—assuming that the acting force, f_1, increases from zero to f_1—is given by

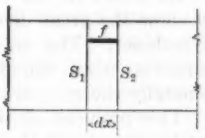

FIG. 15.

$$d\,L = \int \frac{1}{2} f_1 \times e_1 \times d\,A \times d\,x \ldots\ldots\ldots\ldots\ldots (30)$$

Substituting in this the value of e_1 from Equation (29), we have,

$$d\,L = \frac{1}{2} \times \frac{m^2 - 1}{m^2} \times \frac{d\,x}{E} \int f_1^2 \times d\,A \ldots\ldots\ldots\ldots (31)$$

but we know that,

$$f_1 = \frac{M}{I} h \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (32)$$

hence,

$$d\,L = \frac{m^2 - 1}{m^2} \times \frac{M^2 \times d\,x}{2\,E\,I^2} \int h^2 \times d\,A = \frac{m^2 - 1}{m^2} \times \frac{M^2 \times d\,x}{2\,E\,I} \ldots (33)$$

Therefore, the work of deformation, as given by Equation (2), is justified.

In establishing the values of V_A, M_{AC}, and M_O, it has been assumed, for the sake of simplicity, that the beam, A-B, was loaded with the uniform load, w. This, of course, gives, for M_{AC} and M_O values a little higher than the actual ones, but the complexity resulting from assuming the parabolic law of loading would not be justified by the results in a practical application.

In order to leave no doubt as to the correctness of Equations (21), (22), and (23), giving the values of V_A, M_{AC}, and M_O, the method used in deriving them is given, as follows: Considering the general

Mr. Janni.

equation for the maximum moment in a single span, quasi-rigidly supported at its ends, and yielding vertically at one of its supports, and given by theory as

$$M_{max.} = M_A + \frac{V_A^2}{2\,w},$$

and applying it to the span, $A\ C$, Fig. 1, where, $M_A = 0$, we will have

$$M_{AC} = \frac{V_A^2}{2\,w},$$

As

$$M_C = V_A\,l_1 - \frac{w\,l_1^2}{2},$$

the vertical reaction, V_A, at A, will be

$$V_A = \frac{1}{2}\,w\,l_1 + \frac{M_C}{l_1}.$$

As stated in the Synopsis of the paper, the writer's intention was to show the great divergency of results obtained by the two different hypotheses. The use of the first assumption, namely, unyielding supports, when the designer has to deal with several panels, as is generally the case, is erroneous.

This problem, no doubt, is one of the most difficult, and a strictly mathematical solution, taking into account the peculiarities of this material of construction, seems to be impossible. This difficulty, however, cannot justify the silence of the building codes on this subject. A reasonable approximation is always possible, and the practical example in the paper gives an idea.

In view of all these considerations, Mr. Godfrey will see that the writer, in order to find the equation of the elastic line, in this particular case, could not use the customary expression,

$$\frac{d^2\,y}{d\,x^2} = \frac{M}{E\,I},$$

which, as Mr. Godfrey knows, is only approximate, although for everyday work it is reasonably satisfactory, since, as has been stated, it would not have brought out the principal physical phenomenon happening in the deflection of a slab.

From Equations (10) and (11), the writer derives a smaller value for D than from the equations obtained according to Mr. Godfrey's suggestion; this difference between the two results is caused simply by the difference in the conditions of the two deflections of the same beam by the two hypotheses.

In order to be clear, it may be stated that, in imagining the slab to be divided as has been done, two peculiar conditions have been created in those beams: The first is that the beams of System *a* help those of System *b* in carrying the load, and *vice versa*; hence the law of parabolic distribution of loading. The second is that these beams are not

free to deform crosswise; hence, the essential introduction of Poisson's coefficient in the equations. Mr. Jannf.

By taking into account, as far as possible, all the differences between plain, single-beam deflection and ideal, slab-beam deflection, we will get nearer the truth for all practical purposes.

Mr. Godfrey's remark, that the parabolic law of load distribution is enough, because it takes into account already the association of the ideal beam, is not quite correct.

In the case of the flat slab, as in other cases of reinforced concrete construction, the reason for the very small deflections usually observed —or the elastic deformations—must be found in the fact that the material (generally speaking) is being compressed or stretched in two directions, normal to each other, which condition is very favorable to the working capacity of the material, as it prevents important deformations.

It is a fact that Poisson's coefficient is purely theoretical, as far as its value, 4, is concerned, but the writer cannot agree with Mr. Godfrey regarding its unreliability for practical purposes, as some tests have given values which differ somewhat from 4, and this discrepancy is due, undoubtedly, to inevitable errors of readings, instruments, etc., besides the fact that isotropic bodies (and concrete may be regarded as such) cease to be isotropic when they are subjected to stresses in only one direction.*

If, as Mr. Godfrey states, some engineers are unwilling to use this coefficient, even in typical cases, this does not mean that it should be omitted from the theory. Those same engineers should know that they are using it, in any case, when they assume the working stress for steel in shear in relation to the working stress for bending, etc.

The value, 4, of Poisson's ratio is intended for isotropic bodies, it is true, but Mr. Godfrey's attention is called to the fact that, in this case, we have to deal with concrete and steel, and the steel (reduced to concrete according to the assumed value of $\frac{E_s}{E_c}$) is placed in the two directions along which it is consistently assumed that the forces are working. In other words, by reducing properly the steel in concrete, we obtain a solid wholly of concrete in the two directions which are of interest in this problem. This, of course, is an approximation, but, taking into consideration the great difficulties of the problem, the writer believes that it is permissible.

The tension in concrete, as Mr. Godfrey well states, undoubtedly plays its rôle in flat slab deformation; and this is also a reason that Poisson's ratio cannot be ignored in this problem.

The use of Equations (12) and (17), and those for the square slab, can be made by practical designers without caring very much how

* Grashof, "Theorie der Elasticität und Festigkeit," Berlin, 1878.

Mr. Janni.

they have been worked out, as is generally the case with formulas, as far as their mathematical derivation is concerned.

The writer will not discuss the question of priority in the method of treating a slab made up of several beams lying in two directions, as mentioned by Mr. Godfrey. In adopting this treatment, the writer never claimed originality.

Starting with the foregoing assumption, in the case of a flat slab design, the fact that, in a square slab, each middle strip at its middle square unit carries one-half the uniform load is self-evident, and the result given by Equation (12), when $l = l_1$, namely, $\alpha = \frac{1}{2}$, is a check of that equation, rather than a result.

To assume that the end moment, M_p, is two-thirds of the fixed-end condition, as has been done by the writer, is similar to—and with nearly the same degree of approximation—that which is made for fixed-end beams, in order to take care of the one-panel loading condition.

In order to demonstrate this statement, attention is called to the formulas, for instance, for the square slab case. There the numerical coefficients should be $\frac{1}{21.33}$ and $\frac{1}{26.94}$ (for a square slab, however, the writer has thought it expedient to state these values in round figures, namely, $\frac{1}{21}$ and $\frac{1}{27}$, in order to memorize them more easily), and those actual values, of course, must be taken for the following demonstration.

According to the accepted rule, the moments for a beam with assumed fixed ends are computed on the basis of the coefficients, $\frac{1}{10}$ and $\frac{1}{12}$ (respectively in the middle of the span and on the supports). These two coefficients give the following results:

$$\frac{1}{10} + \frac{1}{12} = 0.183 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (a)$$

$$\frac{1}{12} : \frac{1}{10} = 0.833 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (b)$$

and the coefficients resulting from the writer's assumption, remembering that these coefficients enter twice into the calculation of the moments for the same panel, will give:

$$2\left(\frac{1}{21.33} + \frac{1}{26.94}\right) = 0.168 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (a')$$

$$\frac{1 \times 2}{26.94} : \frac{1 \times 2}{21.33} = 0.791 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (b')$$

Mr. Janni.

From a comparison of the foregoing equations, (a) with (a') and (b) with (b'), it will be seen that, even though the writer's assumption was "very rough", it would not be any rougher than that usually made for fixed-end beams. Mr. Godfrey's remark, therefore, may be regarded as a little precipitate.

It is true that a slab supported by girders on all edges does not find many users, but this paper is simply the result of the writer's designing of reinforced concrete floors, built years ago, according to these lines, by the expressed wish of the owner, and it was thought that the results might be useful in giving some hint, as stated at the end of the paper, as to a correct solution for flat slab floor design.

Undoubtedly, as Mr. Godfrey remarks, we are badly in need of serious tests in this line, but the writer is afraid that this will never be accomplished, as business has taken such a firm hold on the whole question.

Mr. Goodrich's contribution shows clearly that he understands fully the spirit of the paper. The writer has had no intention of adding any formulas to the more or less empirical ones already in existence, many of them misleading; but his aim has been to show the great influence of the elastic displacements of the supports on the values of the bending moments.

It has not been without a great deal of hesitation that the writer has undertaken to reply to Mr. Jonson. The only reason which prompted him to devote some time and space to this purpose has been the fact that his silence concerning Mr. Jonson's remarks might be interpreted as an acceptance of his statements.

Leaving to Mr. Jonson the responsibility for his statement that "the moment of inertia is not only a constant, but also a function", the writer must say that, when Mr. Jonson brings up the well-known question of the variability of the inertial moment of the reinforced concrete section, he shows quite clearly that he ignores the fact that this moment of inertia, as we find it, and as it was agreed on years ago, must not be intended for each and every stage of the deflection of the beam.

The position of the neutral axis in the cross-section of a reinforced concrete beam, and, consequently, its moment of inertia with respect to it, are purely conventional, as long as the stresses in the concrete and steel are below or above the stresses calculated, but, as soon as the stresses actually in the beam approach those calculated at that section, the actual position of the neutral axis is very close to, if not coinciding with, that assumed. However, if there is any discrepancy, the usual assumptions, in determining the position of the neutral axis, are on the safe side, and this must be so because the tension in concrete is disregarded.

Mr. Janni.

The most important matter in designing a building of reinforced concrete or any other material is not strictly the exact correspondence between calculated and actual stresses, but, indeed, the safety of the building itself.

Designers know very well that the methods used in designing any kind of a building cannot be of such precision that they can be applied, for instance, to the construction of a Bauschinger's Spiegelapparat measuring $\frac{1}{5\,000}$ mm. of deformation.

Neither is it practical to design and construct buildings for ordinary purposes in the same way as beams and floor panels would be designed and built in a laboratory for scientific research.

Mr. Jonson's attention is called to the work done by the French Commission,* which was formed 16 years ago, and, after 6 years of very exhaustive study, published the results. Incidentally, in the writer's opinion, this work so far has been unsurpassed, either in America or Europe.

Mr. Jonson will find there that the question of the variability of E_c has been thoroughly investigated. He will also find that, in addition to all the causes bearing on this variability, mentioned in that work and repeated now by him, the Committee investigated the variation of this modulus due to the method of concreting (vertical or horizontal, post or beam); or due to the personal equation of laborers, etc.; and, finally, he will find that, after taking into account all possible causes, the value of $\frac{E_s}{E_c}$ was not decided on as a real constant quantity, but it was agreed to assume 8 and 15 as minimum and maximum limits, respectively; and this range of values for $\frac{E_s}{E_c}$ has been generally recognized as right, even after later tests and investigations.

The designer may make the choice—not capricious, but judicious—of the proper value of $\frac{E_s}{E_c}$, according to the circumstances. The misjudgment of the designer or of some building law, in the choice of this value, Mr. Jonson will agree, cannot be imputed to theory.

The question of shrinkage in concrete, as affecting the working stresses in a beam, which, from Mr. Jonson's discussion appears to be a discovery brought up by him for the first time, is also old; in fact, on page 371 of the aforesaid work of the French Commission. the first two questions which the Commission proposed to investigate, were as follows:

First.—*"Quelle est l'importance du phénomène du retrait?"*

* Commission du Ciment Armé: Expériences, Rapports, et Propositions, Instructions Ministérielles Relatives à l'Emploi du Béton Armé, Paris, 1907.

Second.—*"Doit-on en tenir compte dans les calculs de résistance?"* and, on the same page, the following answer is found: Mr. Janni.

"Tout en affirmant le fait incontestable qu'est le retrait, la Commission a proposé d'autoriser les Ingénieurs à n'en pas tenir compte, lorsq'ils pourront justifier qu'il ne saurait en résulter d'inconvénients et il parait en être généralement ainsi pour les supports et les planchers intérieurs des bâtiments dont les dimensions horizontales ne sont pas grandes."

The date of this report, and these quotations, will dispose of any possible doubt as to whether Mr. Jonson, having looked into "the entire theory on reinforced concrete" and having found it "rather crude", has brought out a new and important question.

Mr. Jonson, the writer is sure, must realize that the guesswork advocated by him in predicating the position, the direction, the number, and the size of cracks in a restrained reinforced concrete beam, does not and cannot interest an engineer during a serious technical discussion.

As he is attempting also to introduce a new method of designing, in order to take into account the shrinkage, as he says, it will not be amiss to show that this method is wrong.

To start with, Mr. Jonson says "All calculation of reinforced concrete made on the assumptions that the ratio, n or E divided by E_c, is a constant are unreliable", and rather inconsistently he proposes the use of formulas in which this very ratio appears, with the addition of a certain quantity, m, the introduction of which will be discussed later, and which would be an additional uncertain factor in designing.

Apart from many other considerations, however, it can be stated that, from what Mr. Jonson says and proposes to do, it is clear that he ignores that theory has a formula, as follows:

$$f = \pm \frac{N}{A} + \frac{M}{I} h$$

in which f is the maximum allowable stress; N, the compressive or tensile stress along the beam; A, the sectional area; M, the external moment; h, the distance between the farthest fiber and the neutral axis; and I, the inertial moment of the section.

Applying this formula to a reinforced concrete beam, when it is necessary at all, the stress in concrete and that in steel can be computed, remembering that, for practical purposes and for the sake of brevity, the second term of the second member, for reinforced concrete sections, is replaced by a simpler one.

Assume a 20-ft. reinforced concrete beam, with a section of 24 by 14 in., with 3.36 sq. in. of steel reinforcement; and assume that its length is kept constant, so that it is under the full tension caused by the unyielding of its ends during its setting.

Mr. Janni. As the coefficient of the shrinkage is known to be $\frac{1}{2\,000}$, this beam, after setting, is actually under a tensile strain of

$$\frac{20}{2\,000} = 0.01 \text{ ft.}, \quad 0.01 = \frac{N \times 20}{2.33 \times 144\,000}$$

$$\frac{N}{2.33} = 71.856 \text{ tons per sq. ft., or } 998 \text{ lb. per sq. in.}$$

so that each square inch of its concrete cross-section would be under the enormous tension of 998 lb.

Concerning the steel now, it is observed that the rise in temperature which takes place during the setting of the concrete, will increase the length of the reinforcement. This increase, which is the cause of tension in the steel during the first few weeks of hardening of the concrete, diminishes gradually with the further shrinking of the concrete, and, finally, becomes compression, but its value is negligible for all practical purposes.

As it was assumed, however, that the ends of the beam are absolutely unyielding horizontally, it must be assumed (as a matter of consistency) that the increased length in the reinforcement is kept constant.

Assuming, say, 25° Fahr. as the rise in temperature during setting, and assuming 0.0000067 as the coefficient of expansion for the steel, the tensile stress in the reinforcement will be 5 000 lb. per sq. in.

Now, if the bending moment acting on this beam were 1 195 444 in-lb., we would have 998 lb. per sq. in. compression in concrete, and 19 343 lb. per sq. in. tension in steel.

Adding algebraically these stresses to those found in the case of plain tension, we will have 0 lb. per sq. in. compression in concrete, and 24 343 lb. per sq. in. tension in steel.

Furthermore, with a moment $M = 679\,800$ in-lb., we will have the total stresses in the concrete and steel as follows: minimum tension in concrete 431 lb. per sq. in., and tension in steel 16 000 lb. per sq. in.

These results, which are strictly correct, according to the assumption made (horizontally unyielding beam ends), need no further comment.

Everybody, even those not thoroughly familiar with concrete construction, knows that, in common cases, the unyielding of the beam ends is not possible, due to the elastic deformation of the whole construction, by which, providentially, the entire system takes up a condition of reciprocal adjustment among its various elements.

The absence of any perceptible crack in the great majority of reinforced concrete beams clearly supports this assertion; therefore, it

must be assumed that, if there is any initial tension, it must be quite negligible in ordinary cases. Mr. Janni.

If, however, peculiar conditions of the building make it advisable to take into consideration the tension due to shrinkage, then the theory has already everything that is needed by the designer for this purpose, as has been shown.

In order to prove further that Mr. Jonson's theory is not only unnecessary, but is altogether wrong, attention is called to the following facts:

For the action of the axial tension (or compression) and that of a moment on a solid, may be substituted the action of an eccentric tension (or compression), so that, admitting the necessity seen by Mr. Jonson of taking into account in all cases the effects of shrinkage, the beam is to be regarded as a post (with eccentric negative compression).

Everybody knows that the position of the neutral axis, that is, the value of k, in this case, is given by an equation of the third degree, or, more speedily, by a certain graphical construction, but Mr. Jonson obtains for k an equation of the second degree and one of the first degree (his Equations (2) and (7)), *ad libitum*.

It can easily be seen that the diagram, Fig. 3, is drawn incorrectly, and therefore that the values of k are also incorrect.

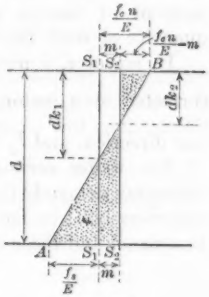

FIG. 16.

That diagram being incorrect (as will be demonstrated), Mr. Jonson's Equation (1) is incorrect, and so are all the succeeding equations which are derived either from his Equation (1) or from Fig. 3.

Let S_1-S_1 (Fig. 16) be the section of the beam under consideration; this section is under the action of a bending moment, the diagram of which may be represented by a straight line, A-B, where S_1-$B = \frac{f_c n}{E}$ is the maximum compressive stress, and A-$S_1 = \frac{f_s}{E}$ is the maximum tensile stress (or their proportional variations of length), and the value of k_1 can be found by the usual methods. (Analytically, the value of k_1 is given by an equation of the second degree.) The axial tensile force, acting on the section, S_1-S_1, will simply bring that section to a parallel position, S_2-S_2.

The diagram, therefore, of these two combined actions, will be represented by A-S_2-S_2-B-A, and the value of k_2, which corresponds to k in Fig. 3, is given, as stated previously, by a third degree equation only. A comparison of Fig. 16 with Fig. 3 will quickly show the mistake.

Mr. Janni.

Having demonstrated that the theory advanced by Mr. Jonson is "fundamentally wrong", and that the real theory is quite able to take care of shrinkage when that is necessary; and after having called attention to the fact that this problem is not new, as it was studied exhaustively 16 years ago, the writer thinks it expedient to disregard all Mr. Jonson's other remarks in this line, for the sake of brevity, and also because the statements and results, being on a false premise, must be wrong, and to reply to his discussion having direct bearing on the paper.

The writer agrees with Mr. Jonson that the moments of inertia cannot be the same in both directions, because the steel is not at the same altitude in the two systems; but if Mr. Jonson thinks that the percentage of steel is very small, and its size also, he will see that this is an approximation which may be allowed. In computing the moment of inertia of a reinforced concrete section, with a small quantity of steel, the steel may be disregarded entirely.

If, however, a more exact result is desired, Equation (12) may be corrected by a factor, $I = \frac{I_1}{I_2}$ where I_1 is the moment of inertia in one direction, and I_2 that in the other.

The writer certainly cannot agree with Mr. Jonson's statement: "economy demands that less reinforcement be used longitudinally than transversely". In fact, the moments at the center of each middle beam in a panel and those on the supports are tied together by the relations:

$$\frac{M_O}{M'_O} = \frac{l^2\,(7 + 43\,\alpha)}{l_1^2\,(50 - 43\,\alpha)}$$

$$\frac{M_P}{M'_C} = \frac{l^2\,(1 + 4\,\alpha)}{l_1^2\,(5 - 4\,\alpha)}$$

As stated in the paper, the value of α may vary between $\frac{1}{2}$ and 1, and inspection of the expressions mentioned shows that the maximum moments are on the shorter span; therefore, a larger quantity of steel must be used in that direction. Thus "economy demands" nothing; it is only a matter of results obtained by theory.

Mr. Jonson says:

"The author's method implies the assumption that all moments are at a maximum when the entire floor is covered by the live load. This assumption is erroneous."

Then he gives the well-known "recipe" for obtaining the maximum moments in continuous beams, which he thinks the writer should have followed in order to be correct.

Mr. Janni.

Evidently, Mr. Jonson ignores the fact that the usual "recipe" does not hold good when the supports are elastic, that is, when they are displaced vertically under the loading. In fact, let Fig. 17 represent the middle beam under consideration in the paper, and suppose that C and D are unyielding; then, assuming that only the span, C-D, is loaded, and applying the theorem of three moments* to the supports, A, C, and D, and remembering that the moment at A is zero, and that the moments at C and D, by symmetry, are equal, we will obtain, with the same unit load as in the application treated before:

$$188 \text{ lb. (live load)} = 300 \text{ lb. (total load)} - 112 \text{ lb. (dead load)},$$

$$M_C = 5\,414 \text{ ft-lb.}$$

and, therefore, the moment, M_O, at the center of the span, C-D, will be given by:

$$M_O = \frac{1}{8} \times 188 \times (24)^2 - 5\,414 = 8\,122 \text{ ft-lb.}$$

If these results are compared with those obtained by the writer, when he took into consideration the yielding of the supports, C and D, as must be done, the value of Mr. Jonson's remark will undoubtedly be realized.

Incidentally, it may be noted that his statement concerning the most critical condition of partial loading is the very thing brought out by applying the theorem of three moments to a continuous beam, although Mr. Jonson states, a few lines before this remark, that "the theorem of three moments cannot be applied to reinforced concrete, nor any other method which deals in deflection components." In this, assuredly, there is also lack of consistency.

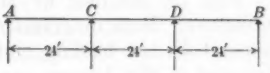

FIG. 17.

It is difficult to understand why, as Mr. Jonson states (the slab being regarded as having fixed edges, and its elastic curve, consequently, being a reversed one), the law of variation of loading "is more closely approximated by a straight line from end to middle than by a parabola"; indeed, it is the writer's opinion that this remark is altogether whimsical. In order to support this statement, however, Mr. Jonson makes the following remark: "The latter [the parabolic law] would be the proper approximation for a non-continuous slab merely supported along its edges".

As a matter of fact, if the parabolic law is the proper approximation for a non-continuous slab, it must be also for a continuous slab. In fact, let A-B-C (Fig. 18) be the parabola of bending moments for the beam, A-C, with the assumption that this beam is supported freely at its ends, and that the parabolic law of loading is adopted.

* The propriety of applying this theorem will be discussed later.

Mr. Janni. If it is supposed now that its ends are fixed, the diagram of bending moments will be obtained, as shown by *A'-B'-C'-E-D*. The line, *D-E*, is to be determined by proper methods, and the parabola, *A'-B'-C'*, is determined by the assumption that the beam, *A'-C'*, is freely supported; therefore, this parabola must be the same as *A-B-C*.

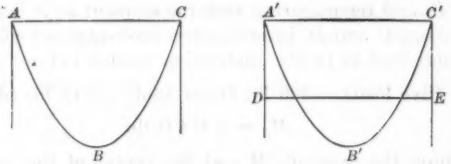

FIG. 18.

This question being settled, on the basis of theory, the writer thinks it expedient, in order to justify his assumption of the parabolic law, instead of the linear law, to show the behavior of the deformed axis of the same beam, subject to both a parabolic-law and a linear-law loading.

Fig. 19, obtained by Mohr's method, gives all the information necessary. Both diagrams represent the same beam, first loaded with a parabolic-law loading and then with the same total load, but distributed, as Mr. Johnson says it should be.

The parabola, *C-O-D*, represents the parabolic law of distribution of the loading, and *E-O-F*, the other assumption of distribution.

By using the same scale in the graphical constructions of the moment diagrams for both hypotheses, by taking the same polar distances in both cases and in each instance, we obtain the same distorted scale for the two diagrams representing the deformed axis of the beam, and, therefore, a comparison between them is made at a glance.

If reliable tests were made, it would be possible to state something definite concerning the degree of approximation of the two assumptions of loading; since, however, such tests are lacking, it may be said that the writer's assumption, already used by others, on the hypothesis of loading, is, at least, as good as that suggested by Mr. Jonson, without substantiating it in any way.

Incidentally, it will be remarked that, in assuming the parabolic law, the writer followed the very method just shown and found no reason to change what already had been done.

Mr. Jonson, it seems to the writer, is rather concerned that the paper, in some points, does not convey his (Mr. Jonson's) ideas, but the writer's ideas, and, accordingly, he attempts to correct what he thinks is erroneous.

Mr. Janni.

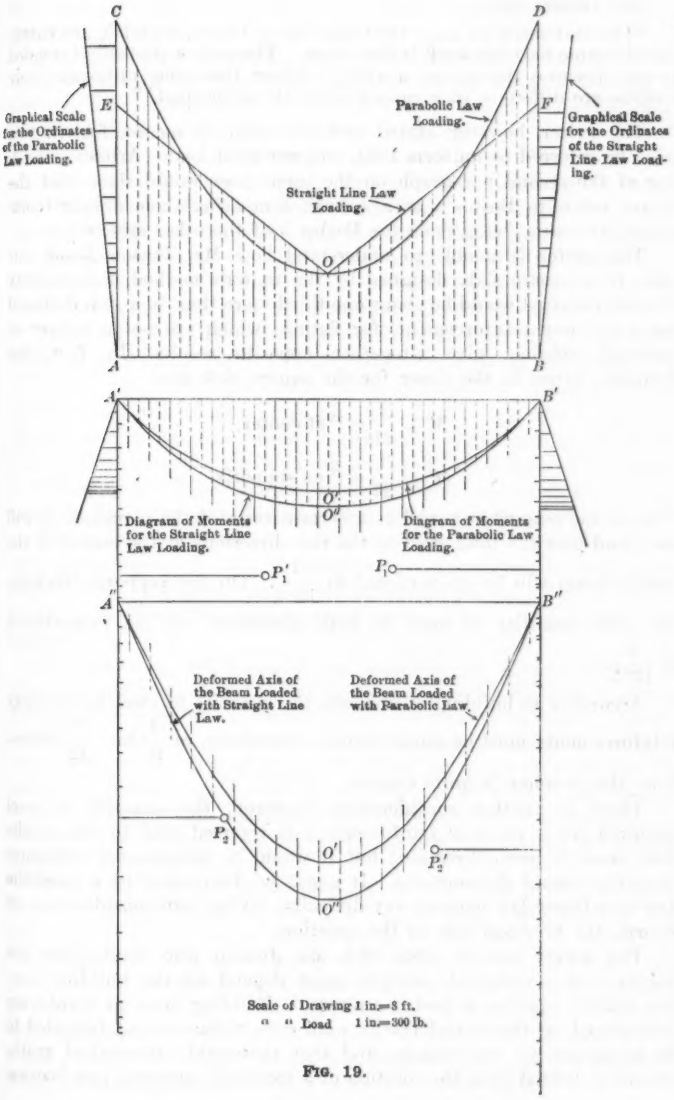

FIG. 19.

Mr. Janni.

Mr. Jonson says:

"The statement on page 1690 that 'these beams, *a* and *b*, are carrying the same uniform load' is erroneous. The author probably intended to say 'because the beams, *a* and *b*, deflect the same, although their lengths are different, they cannot carry the same load.'"

The writer, however, stated correctly what he meant, for, in this case, he referred to uniform load, and not total load. Attentive reading of the second paragraph on the same page would show that the writer refers to Beams *a* as a system, comparing them among themselves; the same being done for Beams *b* of the other system.

The writer is unable to understand how Mr. Jonson found out that, in a square slab, designed by the writer's method, the quantity of steel required would be about one-third more than in a slab designed for a one-way reinforcement, for figures, which are not a matter of personal opinion, give altogether opposite results. In fact, the formulas given in the paper for the square slab are:

$$M = \frac{1}{21} w l^2 \text{ at center,}$$

$$M' = \frac{1}{27} w l^2 \text{ on supports.}$$

Taking the quantity of steel as a certain ratio of the moments, it will be found that the total steel in the two directions at the center of the middle beam will be proportional to $\frac{1}{10.5}$. On the supports, likewise, the total quantity of steel in both directions will be proportional to $\frac{1}{13.5}$.

According to building ordinances, the quantity of steel for one-way reinforcements must be proportional, respectively, to $\frac{1}{10}$ and $\frac{1}{12}$; therefore, the economy is quite evident.

There is another consideration, however; the quantity of steel required for a two-way reinforcement is referred only to the middle 1-ft. beam in each direction; but it would be progressively excessive in going toward the supports. It might be diminished by a parabolic law or a linear law without any difficulty, taking into consideration, of course, the practical side of the question.

The writer cannot agree with Mr. Jonson who thinks that the solution of a technical question must depend on the building law; the writer's opinion is just the reverse. Building laws, as a rule, are formulated on theoretical truths, and, even then, are not intended to be textbooks, by any means; and that thoroughly theoretical truths should be barred from the solution of a technical question, just because

they might conflict with some stale building ordinance limitations, is somewhat ridiculous. Mr. Janni.

By far the most interesting part of Mr. Jonson's discussion is that which deals with a certain graphical construction which, according to his judgment, should be followed, in order to avoid "the illegitimate use of the theorem of three or of four moments."

In order to be clear, the writer will devote a few words to the use of the theorem of three moments, and will demonstrate, incidentally, that its use in the case considered by him in the paper, was quite legitimate. He will then deal with what Mr. Jonson calls "the theorem of four moments" and its quite correct application to reinforced concrete girders; finally, the graphical method proposed by Mr. Jonson will be taken up, with some of its astounding consequences.

Evidently, when Mr. Jonson says that, on account of the variability of the moment of inertia in a reinforced concrete girder, the application of the theorem of three moments is "illegitimate", he either forgets or ignores two fairly important facts: The first is that the theorem of three moments may be extended legitimately to the case in which the moment of inertia of a girder is variable; and this is done by the theory of virtual work. This theorem has progressed somewhat since Clayperon, in 1857, first gave his simplified solution of this problem.

The second fact—and this is the ordinary case in reinforced concrete construction—is that, in the case of supports on the same level, the last term of the equation of the theorem of three moments, which would bring forth the moment of inertia of the cross-section, disappears.

The values of the moments on supports, for girders of various spans, are already calculated in engineering handbooks, and, of course, without any moment of inertia to be substituted in them. Hence, admitting only for a moment that a reinforced concrete girder is freely supported at several points of its length, no matter whether the moment of inertia is constant or variable, and no matter also whether or not its supporting points are on the same level, the equation of three moments may be used legitimately; this is as far as the general theory of this theorem applies.

The writer will now justify the use of this theorem in the paper: It will be noticed that the beam, *A-C-D-B*, rests freely on four supports, since the sections of this beam on the supports, *A* and *B*, are admittedly free to have whatever very small rotation may take place on account of the deflection of their own spans, and since the supports at *C* and *D* are not less loose, according to the temporary assumption, than at *A* and *B*. One should not take seriously the question of "torsional resistance" of the supporting girders at *C* and *D*, especially when, as it is assumed, the beam, *A-C-D-B*, is unloaded.

Mr. Janni.

Furthermore, the theorem of three moments has been used in order to find M''_C, that is, the moment at C when the assumed vertical displacement takes place, and this quite independently of any hypothesis of loading for the beam itself. In other words, having the supports, A, C, D, and B, on the same level, and the beam unloaded, if, for some reason, the supports, C and D, give way vertically, a certain moment will appear at C and D; this is the moment found by the writer by a quite legitimate use of the theorem of three moments.

Mr. Jonson directs attention to what he calls "the theorem of four moments", and gives information concerning a paper presented by him before this Society dealing with the subject. The writer has examined the paper referred to and finds that it deals with the analytical solution of the problem of finding the moments in a girder stiffly connected to its supports. This problem was treated by W. Ritter* in 1900; Mr. Jonson's paper appeared in 1905. It can be added, also, that some diagrams in this latter paper are incorrect. The reader may draw his own conclusions.

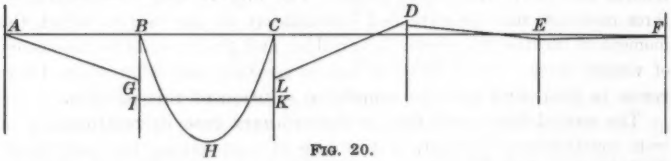

FIG. 20.

A few words will now be devoted to the solution of the problem involved in the case of a girder stiffly connected at various points of its length.

There are two different and mathematically correct solutions of this problem. One is a graphical solution by the method of the ellipse of elasticity, and the other is analytical, following Ritter's theory on this subject, as published in the book quoted.

For practical purposes, the graphical method, which is applied in a manner similar to that used in the case of continuous elastic arches resting on elastic piers†, is (contrarily to what happens in the case of an arch system) to be discarded when the elastic system has more than two spans.

Ritter's analytical method may be followed in every instance. Evidently, the moment developed by the post will be equal to the difference between the two moments, one at the section of the girder immediately on the left, and the other at the section immediately on the right, of the support. This difference between each couple of the similar sections at each support causes, at each of these points, a sudden vertical step in the diagram of the moments, as is shown in Fig. 20.

* W. Ritter, "Der Kontinuirliche Balken", 1900.

† G. A. Hool, "Reinforced Concrete Construction", Vol. III, 1916.

Mr. Janni.

This calculation, in the case of a girder, as it has been assumed, is not a luxury of theory; it is purely a necessity of design. It should be made in order to take into account the real conditions of the elastic system. It is the aim of theory to simplify things as much as is consistently possible, but when a problem involves methods, which, perhaps, are still a little outside of the every-day routine designer's reach, engineers cannot sacrifice their pride to "rules-of-thumb". To disregard the moments in posts stiffly connected to a girder (and usually concrete girders are in this condition when supported by concrete posts) is an error of design which should be avoided.

The last point which the writer will take up is the graphical construction suggested by Mr. Jonson for the design of a three-span continuous girder.

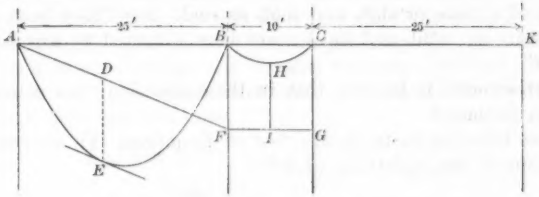

FIG. 21.

Fig. 21 is simply a representation of the graphical construction suggested by Mr. Jonson, and transmitted to the writer by him.

On page 1709, Mr Jonson states:

"The correct method of calculating such a girder is much simpler than by the illegitimate use of the theorem of three or of four moments. Construct a static moment diagram on each span as if it were discontinuous. Then divide the diagram of the long span by a straight line in such a way that the positive moment at the middle of the suspended span is numerically equal to the negative moment in the double cantilever over the column. Then draw a horizontal line representing the negative moment across the middle span, and the area included between this line and the first moment curve will be the true moment diagram. The point at which the first straight line intersects the moment curve of the side span is, of course, the point at which the reinforcement should cross the center line."

It is granted that this graphical solution is "simpler" than the application of the theorem of three moments, or that which takes into consideration the moments due to the post connection; unfortunately, however, this is not a solution, but is something never heard of before. Suppose that the span, *A-B*, remains constant in length, and that the length of the span, *B-C*, increases gradually. According to this graphical construction, the moment at *B* will be always the same (that

Mr. Janni.

is, *B-F*), since the post, *B*, does not move; therefore, no matter how long *B-C* becomes, its moments at its supports will be constant, and their value will be controlled solely by the preceding span, *A-B*.

Mr. Eddy's discussion may be divided into two parts: In the first he states that the fundamental equations of the paper are wrong, since they do not agree with Equations (1) of his discussion, which, as he says, are the equations for a flat slab in their most general form. In the second part he remarks that, since the mathematical working out of the formula is also wrong, the results do not deserve great confidence.

Before proceeding any farther with this part of the discussion, the writer wishes to ask Mr. Eddy whether he is sure that his Equations (1), as interpreted by him, are the correct moment equations for the flexure of a plate or slab, and that, as such, they "have been accepted universally as valid, and have never been disputed by any competent writer?"

Furthermore, is he sure that in these equations due consideration is given to shear?

If we take, for instance, the first of Equations (1), we note that it is the sum of two equations, namely:

$$M_1 = \pm E I \frac{d^2 z}{d x^2}$$

$$- K M_1 = \pm E I \frac{d^2 z}{d y^2}.$$

The first is the differential equation of the elastic line due only to the longitudinal deformation, by compression and tension, in the various fibers of the beam, caused by the moment, M_1; the second equation gives the algebraic additional deformation of the elastic line due to the cross-deformation of the same longitudinal fibers of the beam, as is shown by the presence in this equation of the ratio, K, and of the expression, $\frac{d^2 z}{d y^2}$.

Since, as is very well known, the first of these two equations is quite independent of shear, Mr. Eddy should have devoted a few words to the justification of his statement that, at least, the second equation contains shear. It would be very interesting to see how Mr. Eddy could discover the shear in an equation in which the second member is purely a differential geometrical expression multiplied by a numerical constant, E (which constant refers only to compression and tension), and by a geometrical (or numerical) quantity, I, and the first member contains a certain numerical quantity, K, due to tension or compression, and a moment, M_1, has nothing to do with shear.

Mr. Janni.

An assertion such as that made by Mr. Eddy is very grave, and cannot be passed without being pointed out.

As a matter of fact, Equations (1), with the interpretation given by Mr. Eddy, are quite independent of each other as long as some kind of relation among the expressions, $\frac{d^2 z}{d x^2}$, $\frac{d^2 z}{d y^2}$, is not given. They represent the deflections of two beams which deflect, one under the action of the moment, M_1, and the other under the action of the moment, M_2, and this, contrary to what Mr. Eddy has asserted, is quite independent of shear; therefore, they are not general.

These beams, on account of the same notations, x, y, z, might be supposed to cross each other normally at some point of their spans.

Assuming, as it is legitimate to do, that they be applied to the ideal beams in our case, the writer will show that the fundamental equations of the paper agree with the equations as given by Mr. Eddy, and that the mathematical working out of the formulas, derived from the equations given by the writer, is correct. In fact, taking, for instance, the first of these two equations,

$$(1 - K^2)\, M_1 = \pm\, E\, I \left(\frac{d^2 z}{d x^2} + K \frac{d^2 z}{d y^2}\right)$$

we see that the term, $K \frac{d^2 z}{d y^2}$, of the second member refers entirely to the vertical deflection of the ideal beam, due solely to the lateral (or horizontal) deformation of the cross-section of the ideal beam in that part of the slab.

This lateral deformation has been logically stated to be zero, on account of the working conditions of the ideal beams of System a in connection with those of System b, therefore this equation, applied to the middle beam of one of the two systems, becomes:

$$(1 - K^2)\, M_1 = \pm\, E\, I \frac{d^2 z}{d x^2}.$$

This being the deflection at the end of the cantilever, the work of deformation of the entire cantilever, between the limits, 0 and $\frac{l}{2}$, will be given by:

$$L = \int_0^{\frac{l}{2}} \frac{1 - K^2}{E\, I}\, M \times \frac{1}{2}\, M \,.\, d\, x = \int_0^{\frac{l}{2}} \frac{1 - K^2}{2\, E\, I}\, M^2 \,.\, d\, x.$$

Substituting for K its classical form, $\frac{1}{m}$, we obtain,

$$L = \int_0^{\frac{l}{2}} \frac{m^2 - 1}{m^2} \times \frac{M^2 \,.\, d\, x}{2\, E\, I},$$

which is identical with Equation (2) in the paper.

Mr. Janni.

It is demonstrated, therefore, that Equation (2) is correct, and, of course, agrees with the equations given for this case of deflection. In exactly the same way we might proceed for the other equation of the system given by Mr. Eddy, when it is applied to the other middle beam of the other system of beams. Having demonstrated the correctness of the writer's Equation (2), it follows that Equations (10) and (11) are correct also.

The writer certainly is indebted to Mr. Eddy, as his erroneous remark has given the writer the opportunity of deriving his Equation (2) in an entirely different way.

It is rather interesting to observe that, though Mr. Eddy had found out that "The fundamental equations used by Mr. Janni differ from these [Equations (1) of Mr. Eddy] in reality only in one point, namely, by suppressing and leaving out the last term of each of them", he did not stop to think about the mathematical meaning of this suppression, but confined his investigation purely to a clerical check. He says: "Mr. Janni in his mathematical work has chosen to develop the subject by attempting to modify and apply the equations of Professor Fraenkel for beams. * * *."

This remark is preposterous. As a matter of fact, there are no such equations as "equations of Professor Fraenkel" on this subject, and the writer is very much surprised at the statement.

What Mr. Eddy calls "equations of Professor Fraenkel" are merely the equations on the deflection of a beam in their most general form, derived by applying the law of virtual work, which any good book on the subject reports, just to show that it is possible, by that method, to generalize the equations on deflection, in the same way as the theorem of three moments by this same law has been generalized and made applicable in the case, for instance, of a variation in the cross-section of a continuous beam.

Mr. Eddy, in making this statement, is crediting Professor Fraenkel with a paternity of which the latter never dreamed.

Mr. Eddy says:

"These errors in Equations (1) and (2) are curiously so related, the one to the other, that Equation (3) is the same as would be obtained were the alterations just pointed out made in Equations (1) and (2)."

It has been already demonstrated that Equation (2) is correct, and it will be shown presently that this is the case also with Equation (1).

Among the most remarkable theorems demonstrated by the law of virtual work, there is one, by Castigliano,* called "Theorem on the

* Castigliano, "Teoria dell' equilibrio dei sistemi elastici e sue applicazioni", Torino, 1879. The original Italian work is not available, but there is a French translation.

derivative of work". This theorem has demonstrated the correctness of the expression: Mr. Janni.

$$D = \frac{d\,L}{d\,F}$$

It may be explained as follows: The displacement, D, of a point of an elastic system along the direction of an ideal force acting on this point, is given by the partial derivative, with respect to the applied ideal force, of the work of deformation of the entire system. Incidentally, it can be safely stated that this theorem and the peculiar form of the foregoing expression are familiar to anybody who has had even the slightest acquaintance with the theory of virtual work. It is hoped that this is enough to substantiate the strict correctness of Equation (1), without inserting in these conclusions a couple of pages of theory taken from some textbook.

Mr. Eddy furthermore states that there is no proof that the equation, $f\,\frac{I}{h} = (1 - K^2)\,M$, is given by any known theory. It will be demonstrated that the theory, on the writer's assumptions, gives this equation as correct. In fact, taking the equation, $e_1 = \frac{m^2 - 1}{m^2} \times \frac{f_1}{E}$, already given herein, and observing that $E\,e_1 = f$ and $f_1 = \frac{M\,h}{I}$, we may write:

$$f\,\frac{I}{h} = \frac{m^2 - 1}{m^2}\,M$$

which demonstrates that the general equation for the deflection has been given correctly by the writer. Mr. Eddy can hardly expect to have reported by theory all possible equations for all possible hypotheses; it is very often the task of the engineer to derive equations for each particular case.

Mr. Turner's Equations (1) and (2), with the interpretation given by him, are not "the fundamental equations of extensional stress and strain established a generation ago and accepted by Grashof and all authorities on the subject, since then".

In the first place, the general equations to which Mr. Turner refers are three, and not two, namely:

$$\left.\begin{aligned} E\,e_1 &= p_1 - K\,p_2 \\ E\,e_2 &= p_2 - K\,p_1 \\ E\,e_3 &= -\,(p_1 + p_2)K \end{aligned}\right\} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (34)$$

They are obtained by considering the equilibrium of a small cube (element), two faces of which are always normal to the vertical plane of symmetry of the cross-section of a beam, and, at the same time, are parallel to one of the two "principal stresses" reacting at that point;

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it follows that the other two faces of the cube, will be parallel to the other "principal stresses".

Let A-B (Fig. 22) be the vertical profile of a cross-section of a beam, and O the position (altitude) of its neutral axis.

The position of the elemental cube at this point is shown, as well as the directions of the "principal stresses"* acting on that cube at that point. (If p_1 is assumed to be negative, then p_2 is positive, and *vice versa*.)

It being known that the directions and values of the principal stresses, changeable with the altitude of the point of the cross-section at which we consider the cube, it follows that only at the top and bottom of the section this cube has two faces horizontal, and that it revolves through 45° in passing from O to A, and through an additional 45° in passing from O to B.

Furthermore, it is known that the third equation of this system, which refers to the cross deformation of the elemental cube, is zero only when these equations are applied to the cubical element at the neutral axis of the beam, because then, and only then, $p_1 = -p_2$, or $-p_1 = p_2$. Mr. Turner can easily verify this by examining the circular diagram theory, as written by K. Culmann.†

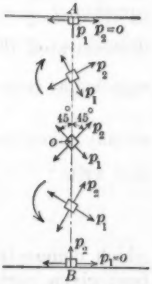

FIG. 22.

Mr. Turner's attention is called to the fact that these stresses, p_1 and p_2, lie on a vertical plane normal to the cross-section of the beam, and not on a horizontal plane, as he states, which is quite different. He will notice, also, by inspecting Fig. 22, that these stresses cannot be made zero where and when he pleases; they are zero only when the circular diagrams, previously mentioned, make them so. These equations, therefore, with the correct interpretations, as given by the writer, are to be regarded as the general equations of equilibrium of a beam, and, as such, they are accepted, of course, by everybody.

Neither Grashof nor any other authority would have accepted the equations, as given and interpreted by Mr. Turner, as general equations for the deflection of a beam, and the use of his illustrious name in this connection is not altogether commendable.

There is also another remark to be made regarding the interpretation of the general equations of the equilibrium of a beam. The writer notes that Mr. Turner applies these "would-be" general equations for this case of equilibrium to a plate, by simply changing the word "beam" to "plate", without thinking that theory cannot be made to apply by simply substituting one word for another. There is great static difference between the equilibrium of a plain beam and a plate.

* These "principal stresses" are sometimes designated improperly "diagonal stresses".

† "Die graphische Statik." Zurich, 1866–75.

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Equations (1) and (2), as given by Mr. Turner, are simply Equations (26) and (27), as given by the writer; that is, the equations of equilibrium (not general) applied to the particular case under discussion, as has been shown already in this closing discussion, with the additional difference that the elemental cube, in this case, is assumed to have always two faces horizontal, no matter at what altitude it is considered. So that, when Mr. Turner says: "if p_2 is zero, as it would be in a beam", he is quite wrong. As a matter of fact, in an ordinary beam (and Mr. Turner discusses this kind of beam) the value of his p_2 is never zero, except at the plane of the neutral axis.

The fact that it is never zero in an ordinary beam, though in the ideal beam, as chosen by the writer, it is logically constantly zero, constitutes the radical difference between the two cases of deflection (common beam and ideal beam).

In considering his case of a plate, Mr. Turner states:

"Assuming that K (Poisson's ratio) is zero, it has been demonstrated that the work done in a circular direction is equal to that done in a radial direction. Now, radial deformations alone determine the vertical position of the cantilever plate, and though the circumferential deformations necessarily accompany radial deformations, they provide a means of storage of energy which is not involved in nor determines the vertical deflection".

In the first place, the coefficient, K, of Poisson is far from being zero, therefore, the resulting demonstration, to which he refers, may be interesting, but its conclusion is untrue.

In the second place, does Mr. Turner really mean what he says about the storing of energy? Does he really mean to say that in the theory of elasticity it is quite possible to have work without the corresponding displacement? In other words, if the writer understands rightly, the expression for the work, going entirely by definition, is given by $L = F \times \delta$, in which L is the work, F is the applied force, and δ is the elastic displacement, Mr. Turner states that, by assuming $\delta = 0$ the quantity, L, is not zero. The writer prefers to believe that Mr. Turner did not mean that; although he said so repeatedly, in different ways.

He says, furthermore: "Now, the relative stiffness of the continuous and simple beam, is known by theory to be as five to one". This statement is not exact; the ratio between the deflections referred to is correct only in the case of a beam rigidly supported at its ends, and one freely supported. In dealing with a continuous and a simple beam, the ratio given by Mr. Turner is not exact, but varies according to the hypothesis of loading of the spans.

Apart from this inexactness in Mr. Turner's statement, however, we can see by the statement itself that he makes comparison with a beam with fixed ends, when it is known that such cases of equilibrium

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are discarded from reinforced concrete design; neither does he state whether, in his "numerous experimental results", he used the partial or total loading.

The writer must confess that he is unable to follow Mr. Turner in his "photographic" conception. It may be, however, that his copper half-tone plate comparison may be very interesting to the reader who knows something of this subject; the writer can see neither the plate nor the screen.

At this point of his discussion, Mr. Turner enters another field: "shearing rigidity and flange rigidity". Judging from his nomenclature for this particular internal stress, it seems that he makes a difference between shearing rigidity and what he terms "flange rigidity". It is difficult for the writer to understand, from this new phrase, "flange rigidity", whether Mr. Turner means the resistance of the flange to shear, or some other internal stress just discovered. However, if he agrees with the writer that, usually, exclusive of compression (tension) and shear in the deflection of a beam, there is no other internal stress to be considered, then the writer will conclude that this "flange rigidity" is nothing but the resistance of the flange to shear.

This being the case—as it must be, according to the elementary principles of theory—the shearing rigidity and flange rigidity are simply plain shear in both cases, and therefore all theorems concerning shear may be applied.

Mr. Turner says:

"Another matter which is very puzzling to many is the fact that the mechanics of a solid is such that ordinary rules of statics do not apply, that is, the rules of statics as applied to the separate members of framed structures are not here applicable."

As an example of this he states that, "under pure statics, a force in one direction cannot affect the magnitude of the force at right angles thereto", but in mechanics "that is not the case", since here the "deformation in one direction under a given force affects and influences the deformation produced by a force at right angles thereto". This—to say the least—shows a peculiar confusion between forces and deformations, as well as a quite new conception of statics.

In the first place, all the rules of statics, being mathematically true, stand, no matter whether they are applied to a separate member or to a framed structure; there is not, and never will be, it can be stated with mathematical certainty, a single problem in mechanics where these rules will not apply, as Mr. Turner wrongly states.

It is true that sometimes the rules of statics are not sufficient for the solution of certain problems, for instance, the elastic arch, the continuous girder, etc., and then help must be obtained outside of

statics; but this does not mean that these rules do not apply. It is no wonder, therefore, that many are very much puzzled by such a statement as Mr. Turner has made. Mr. Janni.

Secondarily, though the example he cites as a proof of his assertion does not mean anything bearing on the assertion itself, it shows an alarming confusion in Mr. Turner's mind between deformations produced by force and by components of that force; between causes and effects.

The deformation produced by the forces, p_1 (Fig. 14), affects that produced by the forces, p_2, but this does not mean that the forces, p_1, are the components of the forces, p_2, or *vice versa*. The forces, p_1, have no partial or total components along the forces, p_2, and this is true here as well as in plain statics; the same may be said of p_2 with respect to p_1.

From the foregoing statement of Mr. Turner, it can be safely said that, to the realization of perpetual motion, is a very short step.

Taking up the matter of distribution of shearing stress, both in a beam and in a slab, Mr. Turner says:

"In the beam horizontal shearing stresses act only in horizontal planes, but in the slab, with its double change in curvature, horizontal shearing forces act in vertical planes as well."

Apart from the fact that the double change in curvature of the slab has nothing to do with the distribution of this internal stress, it may be said that the foregoing statement is altogether wrong. Mr. Turner undoubtedly knows that at any point of the cross-section of a beam there is a horizontal shear and a vertical shear, and these two stresses are equal, this conception being very elementary. How, therefore, can he say that in a beam there is no vertical shear? If he, reversing the entire theory on shear, does not admit this vertical shear, he does not know that without it the internal stresses, taken altogether, cannot be in equilibrium.

Concerning the slab, however, he concedes that there is a vertical shear, but that vertical shear is there on account of double change in curvature. That is equivalent to saying that, if the slab has no double curvature or has more than a double curvature, the vertical shear will disappear.

Whatever Mr. Turner's ideas are regarding this matter of shear, it may be stated that, as a matter of fact, the vertical shear in the slab is always there when there is horizontal shear. In other words, the law of the distribution of shearing stresses, along the horizontal and vertical faces of the elements of a beam, holds good also in a slab, as it must.

It is true that in a square slab, for instance, there are lines along which there is no vertical shearing stress, but, accordingly, along those

Mr. Janni. same lines there is no horizontal shear either; of course, this happens when the slab is assumed to be uniformly loaded.

Mr. Turner, the writer is sure, must know that the theory of shear, holds good for beams, or for slabs, or for arches. This theory is quite independent of the external shape of the solid. The only assumption—made at the beginning—is that the two cross-sections, A-B and A'-B' (Fig. 23), of a solid shall be so near that, even if the whole solid has variable shape, these two sections may legitimately be considered as equal.

The single or double curvature mentioned by Mr. Turner is a deformation brought about by the application to the solid of external forces, and, as such, like all elastic deformations, they have no influence whatsoever on the law of the distribution of the internal stresses of the solid.

Sometimes these deformations, as in the case of arches, bring out additional stresses which are to be added properly to the former ones; but this does not destroy, nor modify in the least, the manner in which the former ones are distributed in the interior of the solid. Therefore, speaking of double curvature of a solid as affecting the law of distribution of internal stresses is erroneous.

From Mr. Turner's remarks concerning the two glued or bolted planks and the cured concrete slab, it appears that he considers all the strength of a beam and of a slab to depend on the shearing stress at the neutral plane of these solids.

FIG. 23.

As a matter of fact, the shear at the neutral plane (as well as that at any other plane) is necessary to the resistance of these solids, but is not more necessary than tension and compression. All these internal stresses have their rôle; all are necessary, but one is not more necessary than the other. Take the case of the two planks; after they are bolted together it is found (correctly) that the resistance of this new beam is twofold, and from this fact Mr. Turner concludes that this is due to the shear obtained along the neutral plane; accordingly, he speaks of its great importance in the resisting quality of the beam. He, however, does not stop to think that, if this new beam is cut across vertically (for instance, at its center) from the bottom up to the glued plane, or is cut with a V-shaped notch from the top down to the glued plane, it, with all its shearing rigidity, is worth no more than one of the planks of which it is constructed, as far as bending is concerned. Is there any reason, therefore, for giving to the shear rigidity so much preference?

The writer has taken up simply the important theoretical points brought up by Mr. Turner, but has not discussed either his criticism of the paper, or his eulogy on a certain theory on flat slabs written by Mr. Eddy.

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As Mr. Turner has not shown any particular care in his quotations from theory, and has not been clear in his conceptions of the strength of materials, the writer is of the opinion that his criticism of the paper has no bearing whatever.

Therefore, if the "most gratifying concordance between deflections and stresses", as Mr. Turner states, has been found on the strength of the kind of theory quoted by Mr. Turner in his discussion, is there any "more cogent reason" for stating that the theory he is supporting is not a theory but merely ordinary guesswork, and as such should be objectionable, for it is unsafe?

Will engineers dump and bury forever a public patrimony, called theory, in order that Mr. Eddy's theory may stand up?

No one will have any hesitation in his choice.

In reply to Mr. Mensch's remark, that the writer's theory is misleading because the moments calculated by it are not the same for all strips, the writer wishes to state that he never meant that the reinforcement in the entire slab should be computed according to those values. It is stated in the paper that the intention is to calculate the maximum in each direction only, and for this reason alone he selected the middle strips.

Knowing the maximum values of the moments at the center of the middle span of each middle beam, and their values at the support points, the arrangement of the reinforcement is an easy matter.

Since the remaining part of Mr. Mensch's discussion, like a good part of several of the others, seems to be a discussion of Mr. Jonson's remarks, to which the writer already has answered at length, it is thought expedient to disregard that part in order not to present, practically, the same arguments.

Mr. Marsh brings out the fact that Equation (12), for ratios of $\frac{l_1}{l}$ greater than $\frac{5}{3}$, gives a transverse coefficient greater than unity and a minus quantity for the longitudinal coefficient. This he points out as a defect of this formula.

Several formulas in engineering mathematics, as Mr. Marsh undoubtedly knows, need logical interpretation before proceeding to apply them.

Take, for instance, the very formula in which the coefficient of Poisson has its origin, that is:

$$\left(1 + 2\,\frac{e}{m} - e\right) A \,.\, \Delta x = V'$$

where: e = stretching of the fiber per unit of length;
m = coefficient of Poisson;
A = area of the cross-section of the fiber;
Δx = its length;
and V' = volume of the stretched fiber.

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In order to obtain our result, we must observe first that the volume of the stretched element cannot, logically, be increased by the actions of the two tensile forces applied to it; accordingly, we are able to put the condition that $\frac{e}{m} < \frac{1}{2} e$, from which we derive that the value of $\frac{1}{m}$ is between 0 and $\frac{1}{2}$. Without this logical assumption, we would never be able to determine the limits of $\frac{1}{m}$. Indeed, we might come to the absurd conclusion that the volume of the fiber, $A \cdot \Delta x = V$, increases under the action of extensional forces.

The formula of Euler is another instance. Mr. Marsh undoubtedly knows that the indiscriminate use of this formula may lead to absurdity, due to its nature, as it is not a purely mathematical formula but one of elasticity.

A formula of elasticity is to be regarded as absurd when at each and every stage of its values it brings out absurd results, or else it is derived on absurd assumptions.*

The writer, however, is indebted to Mr. Marsh for having pointed out this fact, as it has thus given an opportunity to explain fully this point also.

Mr. Marsh remarks also that the introduction of Poisson's ratio is doubtless essential to this problem, but he has misgivings regarding its practical utility and accuracy.

It can be pointed out that the little justified doubt concerning the accuracy of the coefficient of Poisson does not destroy the correctness of its introduction in this problem. There is an incontestable physical phenomenon, which happens in this case of deflection, and as this was recognized as essential by the writer and also by Mr. Marsh, it must be taken into account.

The omission of the coefficient of Poisson in any flat slab theory, means the undermining of that theory.

The writer regrets that Mr. McMillan has devoted his valuable time entirely to the discussion of the points brought out by Mr. Jonson. It is true that if those points had cast a new light on the whole theory of reinforced concrete they would have undermined also the solution of the problem proposed and solved by the writer, but Mr. McMillan knows very well that those points were not new by any means, and that their solution was already a matter of fact, and this apart from the fact that the solution proposed by Mr. Jonson lacks any serious mathematical support, as has been shown.

* In pure mathematics we have a similar state of affairs in the solution of equations above the first degree. Mathematically speaking, all the roots of an equation are true and solve the same equation; but how many of those roots must we discard because they lead to absurd conclusions in our problem? Does this mean that the theory of the solution of equations is wrong?

Mr. Janni.

As a general impression of the various discussions on this paper, it can be said that only those by Mr. Goodrich, and by Mr. Godfrey took up the important point brought forth by this problem, namely, the great variation of the moments in a continuous slab determined by the yielding of the supports.

It seems to the writer that attention has been centered only on the formulas for the design of a flat slab rigidly supported, which formulas, after all, were purely incidental to the problem.

Apart from the fact that these formulas, as far as the difficulty of the problem has allowed, have been demonstrated to be correct both in their conception and from a mathematical point of view, there is the fact that, even applying other formulas, theory would have given the same results, as far as difference of moments in both hypotheses is concerned.

The writer, whose professional activity is not narrowed down to the designing of reinforced concrete flat slabs, was unaware that there could be such a thing as "A flat slab specialist", but, after this discussion, he must say that, if there is such a specialty, its theory needs some decided improvement.

In this closure the writer cannot refrain from expressing his opinion concerning technical discussions.

There is no doubt that a discussion on a scientific question is more apt to bring to light important truths than an ordinary publication on that subject, and, in the opinion of the writer, a sound, well-thought-out contribution to the subject in question is not only welcomed by the interested public, but more so by the author of the paper himself. Remarks and opinions on a technical problem have a bearing upon it, if they are supported either by theory or by logic, but when they are, so to speak, supported only by a sadly crippled or misunderstood theory, such discussions then become useless and somewhat ridiculous.

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Paper No. 1375

THE PRESERVATION OF SANDY BEACHES IN THE VICINITY OF NEW YORK CITY*

By Elliott J. Dent, M. Am. Soc. C. E.

With Discussion by Messrs. Lewis M. Haupt, Charles H. Higgins, Allen Hoar, F. William Schwiers, A. W. Buel, B. F. Cresson, Jr., and Elliott J. Dent.

Synopsis.

The object of this paper is to set before the Society the result of the writer's study of the effect of wave action on the sandy beaches skirting the south shore of Long Island and the New Jersey shore north of Asbury Park, to call attention to the means by which improved beach property may be protected, and to emphasize the damage that must inevitably result to the beaches as a whole if the erection of structures that interfere with littoral drift is allowed to continue.

The paper treats of such subjects, only, as are considered of primary importance in the particular locality under consideration. It is subdivided into sections, and the contents and conclusions are described by sections:

Tidal Currents and Ocean Currents.—These currents are of no effect except in the vicinity of inlets, and receive only brief mention.

Waves.—The behavior of deep-sea and shallow-water waves prior to the plunge receives a brief mention. The phenomena accompanying

* Presented at the meeting of June 7th, 1916.

the plunge, the up-rush and the back-wash of the wave, and the phenomena accompanying the undertow are discussed and illustrated in some detail.

The Origin of Beach Sand and Gravel.—A brief discussion is given, and the conclusion is reached that the principal source of such material is not the grinding up of cliffs, etc., but the sorting out of particular sizes from the soil formed by the decomposition of the primary rocks.

Typical Beach Forms.—Typical beach forms are discussed and illustrated, particular emphasis being placed on the distinction between the temporary berms and the more permanent dunes, etc.

Constructive and Destructive Power of Waves.—The transporting power of the up-rush, back-wash, and undertow are discussed. The conclusions are arrived at that wave action always causes a transfer of material from the beaches to deep water, that in the normal case berms are constructed during stormy weather and eroded by undermining during quiet weather.

Littoral Drift, and Jetties and Groins.—Littoral drift is described, and the effect thereon of inlets, jetties, and groins is discussed. The conclusions are reached that, for beaches such as those under consideration, inlets, jetties, and groins interfere with littoral drift and increase the quantity of material permanently lost to the beaches.

Bulkheads.—The proper functions of bulkheads and sea walls, their proper location with respect to the berms, and the absence of extensive erosion due to back-wash, are discussed. The conclusions are reached that such structures, when their strength is commensurate with their exposure, will afford a real protection to lands in their rear, that their existence will not prevent the formation of berms when other conditions are favorable, that if not placed too far seaward they will not interfere with littoral drift, and that when used to extend the uplands beyond their natural limits such structures may cause a serious loss to the beaches to their leeward.

The increasing value of ocean beach property has led many investors to build nearer and nearer the shore line, relying on the erection of shore-protection works to guard their investments against damage from the sea. Under-estimates of the forces to be resisted and over-

estimates of the efficacy of certain types of shore-protection works have resulted in extensive property losses in the immediate vicinity of New York City. In view of the importance of such beach colonies as Coney Island, the Rockaways, and Long Beach, on the south shore of Long Island, and in view of the existence of similar colonies extending in a nearly continuous line along the New Jersey coast from Seabright to beyond Asbury Park, the writer has been tempted to prepare this paper, setting forth such of the general principles of beach formation and erosion as apply to these localities.

The first time he was required to report on plans for the protection of a portion of one of these beaches, he advocated the erection of a system of groins. The direction of the littoral drift was well established, and, in the literature on the subject, no doubt was expressed as to the efficacy of such structures "when properly designed". The case in point appeared to be so simple that the writer had no doubt as to his ability to design the structures properly.

Subsequently, the writer spent much of his time for two summers walking the beaches around New York City. Occasional beach trips were made during the winter, and certain beaches around San Juan, Porto Rico, with which the writer was familiar, were re-visited during the summers of 1914 and 1915. At the same time the literature on the subject was quite thoroughly examined. Memoranda made during these visits furnished the foundation of observed facts on which the discussion herein presented has been based.

There are certain general principles of wave and current action that apply to all beaches, but in one instance the controlling force may be very different from that in another. An effort will be made to state quite fully the principles in regard to the major forces at work on the sandy beaches around the entrance to New York Harbor. Some of the forces emphasized in this paper may be of little or no importance in some other particular locality; and phenomena which, in that other locality, may be of controlling importance may be entirely ignored in this paper. As an illustration, it may be stated that the problem of shore protection at the foot of the caving bluffs along the north shore of Long Island is totally different from that to be faced along the sandy beaches of the south shore; the latter is discussed in this paper.

Tidal Currents.

On an open coast, far away from any inlet, there is a semi-daily landward and a semi-daily seaward current. If the tide is rising at the rate of 1 ft. per hour, and if the mean depth is 1 ft. at a point 3 600 ft. off shore, there will be a shoreward current at that point having a mean velocity of 1 ft. per sec. As the slope of the foreshore, in the locality under consideration, is much greater than 1 in 3 600, the subject of currents due to the flood and ebb tides may be dismissed, except in the vicinity of inlets.

Ocean Currents.

The Gulf Stream directly east of the mouth of Chesapeake Bay has a surface velocity of about 2½ ft. per sec. If a steady current of equal velocity flowed along the New Jersey coast, in shallow water, with a loose sandy bottom, we might expect to find sand waves, deep pools, and shallow bars, all subject to frequent changes in position. Such conditions do not exist along the shores under consideration, and, so far as this paper is concerned, no discussion of ocean currents is necessary.

Waves.

Wave action is an important factor in the construction or destruction of ocean beaches. A few of the general principles will be given here, and the reader may consult the various works on the subject if further information is desired. The writer has followed the paper "Wave Action in Relation to Engineering Structures".*

The form of a wave is normally that of a common or prolate cycloid. If we assume several filaments of water, parallel to the direction of travel of the wave, and at certain selected distances below the surface of the water, we shall find that there is no tendency of these filaments to cross as the wave passes by. The passage of a wave does not mix the surface water with the deeper water, or *vice versa*.

In deep water each particle of water coming within the influence of a wave moves in circular orbit. (Fig. 1.) The orbital velocity is uniform, and in the upper half of the orbit the motion is in the

* By the late Col. D. D. Gaillard, Corps of Engineers, U. S. Army, published as Professional Paper No. 31, Corps of Engineers, U. S. Army.

direction of wave travel. When the wave has passed, each particle is left in the position it occupied before the wave arrived. The radii of the orbits decrease with increases of depth below the surface.

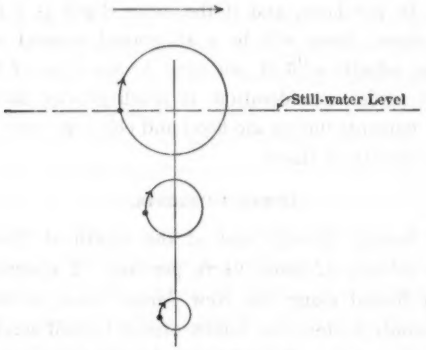

FIG. 1.—AS A DEEP-WATER WAVE PASSES, EACH PARTICLE OF WATER MOVES IN A CIRCULAR ORBIT; THE RADII OF THE ORBITS DECREASE WITH INCREASED DEPTH BELOW THE SURFACE; WHEN THE WAVE HAS PASSED, EACH PARTICLE IS LEFT IN ITS ORIGINAL POSITION.

In shallow water (Fig. 2) the orbits are elliptical in form. The semi-axes of the orbits decrease with increases in depth below the surface, but the focal distance of all orbits is constant. The horizontal or major semi-axis approaches one-half of the focal distance as a limit. The vertical or minor semi-axis approaches zero as a limit. A particle of water very near the bottom moves in what is, to all intents and purposes, a straight line. It is important to note the orbital velocity of such particles.

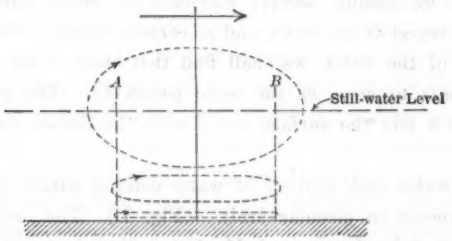

FIG. 2.—IN A SHALLOW-WATER WAVE, THE ORBITS OF THE VARIOUS PARTICLES OF WATER ARE ELLIPSES; THE FOCAL LENGTH OF ALL ORBITS IS A CONSTANT, *A B*; NEAR THE BOTTOM THE ORBITS APPROACH A STRAIGHT LINE AS A LIMIT; THE VELOCITIES ACCOMPANYING THE FORWARD AND BACKWARD SWINGS ARE EQUAL.

If a circle is described, concentric with the elliptical orbit, and with a diameter equal to the major axis of the orbit; if a point moves along the circumference of the circle with a uniform velocity, such that the time required to complete the circle is the same as that required by the particle of water to complete its orbit, then the orbital velocity of the particle of water will be such that it will at all times lie vertically above or below the point on the circumference of the circle.

The orbital velocity of the particle of water is at a maximum as the trough or the crest of the wave passes over it. As the trough passes, the orbital motion is opposite in direction to the travel of the wave; as the crest passes, the orbital direction is with the wave travel. The velocities in the direction of wave travel and in the opposite direction are equal.

With a moderately sloping foreshore, deep sea waves approaching the beach are diverted in direction. The tendency is to change direction until the travel is perpendicular to the beach. Waves seldom strike the beaches under consideration at an angle of 45° from the perpendicular, or greater.

Assume for the moment that one is observing the effect of a heavy swell following a storm. In deep water a large rounded wave would be seen, its surface section corresponding approximately to the curve of a prolate cycloid; as the water becomes more shallow, the wave rises higher, becomes more pointed, and the section corresponds more closely to that of a common cycloid. When a certain depth is reached, the wave breaks and the upper point falls over, striking the front slope. Depending on the character of the bottom, the magnitude of the wave, etc., this first break may occur in depths exceeding 20 ft.

After the first break, a new wave is formed and travels shoreward until it in turn breaks when it reaches a certain depth. This process continues until water only a few feet in depth is reached, and then the wave breaks for the last time. (Fig. 3.)

This final break is called the plunge, and the point at which it occurs is called the plunge-point.* When the plunge takes place, orbital motion ceases, and a boiling, foaming, sheet of water rushes

* The terms "plunge", "up-rush", and "back-wash" are borrowed from "Coast Erosion and Foreshore Protection", by John S. Owens and Gerald O. Case, Assoc. M. Am. Soc. C. E.

up the beach. This will be referred to as the up-rush. As the wave flows up the beach, the boiling diminishes until a relatively quiet sheet of water has been formed. When the kinetic energy has been entirely wasted, or transformed into potential energy, due to the increase in elevation, the back-wash begins, and the water flows down the slope in a smooth sheet. The velocity during this back-wash constantly increases, and, except for a few feet near the upper margin, the back-wash has, in sand, a considerable transporting power.

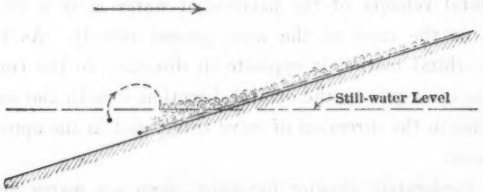

FIG. 3.—WHEN THE PLUNGE OCCURS, ORBITAL MOTION CEASES, AND A BOILING MASS OF WATER RUSHES UP THE BEACH; WHEN THE KINETIC ENERGY HAS BEEN ENTIRELY DISSIPATED, OR TRANSFORMED, THE BACK-WASH BEGINS, AND THE WATER FLOWS DOWN THE SLOPE IN A SMOOTH SHEET; THE TRANSPORTING POWER OF THE UP-RUSH ON THE UP GRADE MUST, FOR EQUILIBRIUM, BE EQUAL TO THE TRANSPORTING POWER OF THE BACK-WASH ON THE DOWN GRADE.

On sandy beaches, the height of the wave at the time it reaches the plunge-point is not great. A height of 5 ft. is not infrequent, but the writer does not recall having seen breakers as high as 8 ft. The depth of water corresponding to the plunge is seldom more than 3 ft.

An after-effect of the plunge that requires special notice is the undertow. When the back-wash reaches a certain point, normally about the line of the still-water level, a new wave is encountered. The water from the earlier wave must find some escape to sea, and it is a matter of common knowledge that an off-shore current, hugging the bottom, is formed. This current is known as the undertow.

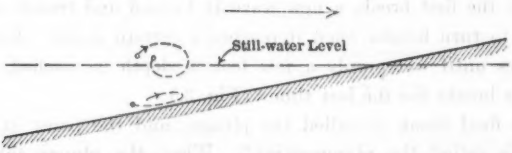

FIG. 4.—NEAR THE SHORE, AS A RESULT OF THE UNDERTOW, THE CLOSED ORBITS ARE REPLACED BY SPIRALS; NEAR THE SURFACE, THERE IS A PROGRESSIVE LANDWARD MOVEMENT, AND NEAR THE BOTTOM THERE IS A SEAWARD MOVEMENT.

Where the undertow exists, therefore, there must be a new condition of wave action, as shown by Fig. 4. Instead of moving in

closed orbits, the particles of water move in spirals. Near the surface there is a progressive movement shoreward, and near the bottom there is a corresponding movement seaward. If, where there is a good surf running, a bather will raise his feet from the bottom and float on the surface, he will almost invariably drift toward the beach. If he is standing on the bottom, the nearly constant pressure of the undertow will frequently outweigh the occasional thrust of the wave and a considerable effort may be necessary to avoid being carried seaward. The undertow normally concentrates into streams of considerable velocity, and in such places the lower spiral of Fig. 4 is replaced by a current of varying velocity, but always flowing seaward.

The foregoing discussion of waves has been somewhat idealized. Waves ordinarily approach the shore from a diagonal direction, and the undertow leaves the beach by a diagonal on the opposite side of the normal. The undertow may concentrate into a stream nearly parallel to the beach and break out to sea at intervals. Such phenomena will be observed by any one making a study of a particular beach, and it will readily be understood that they are modifications of the idealized case.

The Origin of Beach Sand and Gravel.

The grinding away of rocky headlands and the accompanying reduction of ledge rock to gravel and sand is frequently referred to in print. Similarly, the wearing away of boulders as they are carried along by mountain torrents is a constantly recurring theme, attractive to the imagination. If, however, the prosaic engineer will pause for a moment to consider, it will be apparent that these are relatively unimportant sources from which beach sand is derived.

The constant agitation of a gravel beach must inevitably wear away the stones and gradually reduce their size. The rounded form of the pebbles is due to this wearing process. Each pebble is constantly reduced in diameter by the conversion of its outer skin into dust, but, by the time any given quantity of gravel can, by the attrition process, be reduced to grains of sand, there will remain only a small percentage of the original volume.

The principal source of sand in America is the verdure-covered mountain, hill, and valley. Wherever rain falls and vegetation grows the primary rocks are undergoing a process of disintegration. Certain

components are carried away in solution, other constituents are transformed into clay, and the quartz crystals remain nearly unchanged. Let some of this decomposed rock, which is called soil, fall into a stream of running water, and the various sizes are quickly sorted out. The clay may be carried to deep water in the ocean without a halt, the finest sand may travel hundreds of miles before it is temporarily deposited, the coarser sand may be deposited within a few miles, and some fragments of rock may be rolled only a few feet during each freshet, and may, in traveling a few miles, be converted into typical rounded pebbles.

The nature of the sand found in most rivers and on most beaches is proof of its origin. It is the friable but insoluble quartz which has survived, and the grains are much more angular in form than pebbles formed by the attrition process. In the immediate vicinity of rocky cliffs, exceptions may be noted, and the writer has seen large areas covered with volcanic sand, but the beaches to which this paper particularly refers are no exceptions to the general rule.

Typical Beach Forms.

If all except wave action could be eliminated, the normal form of beach would be a series of berms. Several such berms are sometimes found in the case of gravel beaches. In the case of sand beaches, two berms are often noticeable.

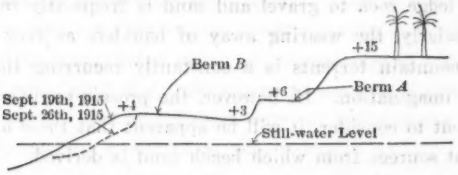

Fig. 5.—Beach at San Juan, Porto Rico, September, 1915. The Cocoanut Grove Stands on Semi-Permanent Sand Dunes; Berm *A* Represents the Remains of a Much Wider Berm Built up During the Preceding Winter and Eroded Back to the Scarp During the Summer; Berm *B* was Formed During a Storm, about September 15th, 1915.

Fig. 5 shows the general character of a beach at San Juan, Porto Rico, as noted on September 19th and 26th, 1915. This beach builds up every winter during the stormy season. During the summer or quiet season, it cuts away, the limit of cutting being indicated by the location of the cocoanut grove. Berm *A*, at Elevation + 6, was formed during the winter of 1914-15. By September, 1915, the beach had cut away as far as the upper scarp which, in September,

1915, marked the limit of that berm. During a 3-day storm, about the middle of September, a new berm, *B*, was built up in front of Berm *A*, and at a lower level. During relatively quiet weather prevailing from September 19th to 26th, the lower berm was eroded by undermining, as indicated by the broken line. The high bank on which the cocoanut grove stands represents a relatively permanent formation; the lower berms are temporary in character. Almost identical conditions have been observed at Long Beach, Long Island, on numerous occasions, and on one occasion four berms were noted in a gravel beach at Mattituck, Long Island.

Information as to the form of beaches below the low-water line is not as complete as could be desired. In general, there is some depth at which the beach quite suddenly assumes a much flatter slope than it has at and above the level of mean tide. Around New York City, with a mean tidal range of about 4.5 ft., this change appears to come at about the level of mean low water. At San Juan, Porto Rico, where the tidal range is about 1.1 ft., the change of slope is very sharp, and occurs in water from 2 to 3 ft. deep. A beach noted by the writer on the Island of St. Thomas was similar to that at San Juan.

Constructive and Destructive Power of Waves.

Just before the plunge, a wave, by virtue of the orbital velocities of its component parts, possesses a considerable store of energy. After the plunge, this energy is largely dissipated, due to the turbulent flow accompanying the up-rush. To preserve the equilibrium of the beach, the turbulent up-rush must have a sand-carrying capacity on the up-grade equal to that of the back-wash on the down-grade. Shoreward of the plunge-point, the constructive power of the up-rush is greater than the destructive power of the back-wash and undertow. The slope of the beach, taken in connection with the character of the beach material, is an indication of the extent of this difference. It would seem that, seaward of the plunge-point, the destructive agencies are more powerful than the constructive agencies. Referring to Fig. 4, it is seen that, near the bottom, the resultant movement is seaward. Unless the sand can be carried to the top of the wave, there is no force to carry it shoreward. The ultimate effect of wave action, therefore, is a constant transfer of sand from the beach to deep water in the ocean.

As already stated, there is, in the beaches near New York City, a change in slope near the line of mean low water. Above that line, the slope is the measure of the difference in transporting power, for the particular material under consideration, of the up-rush and the back-wash of the wave. In coarse sand at San Juan and at Seabright, N. J., this slope averages perhaps 1:12; at Long Beach, N. Y., with fine sand, the slope is perhaps 1:40. For any locality, this slope is nearly constant, regardless of the magnitude of the waves that have created it and regardless of whether it is the result of an addition to, or a denudation of, the beach. Seaward of the foot of this slope, the inclination of the bottom is much less.

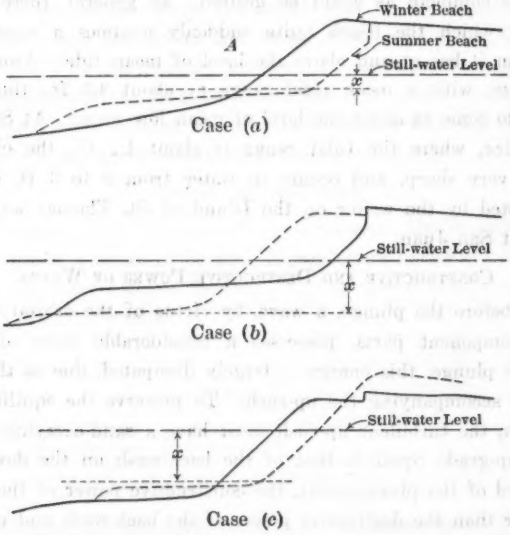

FIG. 6.—THE DEPTH OF WATER AT THE PLUNGE POINT IS INDICATED BY THE DIMENSION, x; THE FORM OF THE BEACH BEFORE IT IS ATTACKED BY WAVES PLUNGING AT THE DEPTH, x, IS SHOWN BY THE FULL LINE, AND THE NEW FORM ASSUMED IS SHOWN BY THE BROKEN LINE.

CASE (*a*) ILLUSTRATES THE EROSION OF A BEACH BY WAVES OF LESS MAGNITUDE THAN THOSE BY WHICH THE BEACH WAS BUILT UP.

CASE (*b*) ILLUSTRATES THE UPBUILDING OF A BERM BY HEAVY WAVES WHEN THE STILL-WATER LEVEL IS AT ITS NORMAL ELEVATION.

CASE (*c*) ILLUSTRATES THE RE-ADJUSTMENT OF THE BEACH FORMED IN CASE (*b*) WHEN ATTACKED BY WAVES OF THE SAME MAGNITUDE AS THOSE ASSUMED IN CASE (*b*), BUT STRIKING THE BEACH AT A TIME WHEN THE STILL-WATER LEVEL IS ABNORMALLY RAISED.

An explanation of the observed fact that berms, built up during stormy seasons, are eroded by the less violent wave action of the calmer

seasons, and that berms attacked by storm waves may be either built up or eroded according to circumstances, may be obtained from Fig. 6, in which three typical cases are indicated.

Case (*a*) represents conditions such as exist when the winter berms are attacked by the summer waves. The plunge-point is at some depth, *x*, and this is the point of maximum disturbance. Shoreward of the plunge-point, the waves maintain a nearly fixed slope, as described in the first paragraph on page 1796. Seaward of that point, the slope of the bottom is steeper than required for equilibrium. The result is a transfer of sand from the berm to the under-water section, and the summer beach indicated by the broken line is formed.

If now, Case (*b*), waves of greater magnitude attack the summer beach of Case (*a*), the plunge-point, in the average case, will be moved seaward to water of some depth, *x*. Shoreward of the plunge-point the slope of the beach is flatter than required for equilibrium, and a transfer of sand toward the beach commences. The ultimate result is a new berm such as shown by the broken line. It should also be noted that at the plunge-point there is also a transfer of sand seaward, and, as this sand is carried into deeper water than that where it lay in the summer beach, the probability of its eventual return to the beach has been diminished. In other words, though the berm has been rebuilt, a certain quantity of beach material has been wasted in the process.

In Case (*c*) it is assumed that the beach formed under Case (*b*) is, in turn, attacked by waves plunging in water of the same depth as in Case (*b*), but that the attendant circumstances are such that the still-water level has been raised. Under these conditions, the berm will be built to a higher level, but the sand used in this elevating process is taken from the face of the berm itself, and the width of the berm is correspondingly decreased.

Many other conditions might be assumed and discussed, but those chosen will show why the beaches around New York City, throughout the summer, present to the eye of a visitor a scarp indicative of erosion; they will show why, in many cases, if not in the majority, the berm at the beginning of the summer is wider than it was at the close of the preceding summer; they will also show why the maximum damage to beach structures usually occurs during an extraordinarily high tide.

The foregoing refinements might have been omitted, and the writer might have limited himself to a brief statement to the effect that, above a certain hydrographic contour, the waves exercise a powerful scouring effect on the bottom; that the sand composing the bottom is constantly agitated and can find no rest until it has reached deep water, or until it has been cast on the beach above high-water mark; that the greater the magnitude of the waves, the wider the zone of agitation, the higher the beach to which part of the material escapes, and the greater the depth to which other material is carried.

Along the south shore of Long Island, the waves, for the greater part of the year, beat on the shore from a direction slightly east of the normal to the shore line. This shore is protected from the northerly storms, and, if the northerly winds are eliminated, the remaining storms will come mainly from an easterly direction.

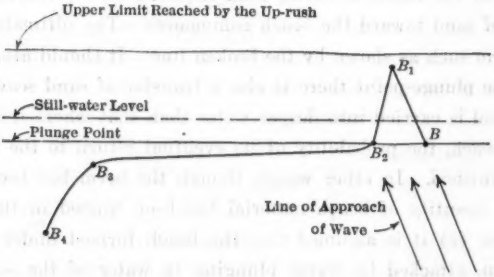

FIG. 7.—LITTORAL DRIFT IS MAINLY DUE TO UP-RUSH, BACK-WASH, AND UNDERTOW. A PARTICLE OF SAND AT B IS DISTURBED BY THE PLUNGE AND CARRIED BY THE UP-RUSH TO B_1, BY THE BACK-WASH TO B_2, AND BY THE UNDERTOW TO B_3 AND B_4.

When a wave strikes the beach from a diagonal direction (Fig. 7), the up-rush carries sand along with it in a diagonal direction. The back-wash also follows a diagonal course, but on the opposite side of the normal. The undertow frequently concentrates into a stream nearly parallel to the shore line, flowing out to sea at irregular intervals.

With the assumed conditions, a particle of sand disturbed by the plunge at B might be carried up the beach as far as B_1. At B_1, the force of the up-rush having been nearly expended, the particle is dropped until the returning back-wash has acquired sufficient velocity to set it once more in motion. The back-wash carries it as far as B_2, where the undertow takes charge. The latter is assumed to be nearly

parallel to the beach as far as B_2. At B_3 the undertow turns seaward, and the particle is carried into deep water.

The course followed by any individual grain of sand is an interrupted one, but, in the normal case, certain particles are at all times following each phase of the route shown in Fig. 7. The majority of the sand grains actually make many journeys up and down the slope, with a progressive movement to leeward, before they are finally taken to deep water by the undertow, or are carried to the semi-permanent uplands above the reach of the waves.

A study of the geology of Long Island indicates that during the glacial epoch enormous quantities of sand and gravel were deposited near the easterly end of the island, in a position where they were subject to wave attack. The prevailing direction of the waves has caused part of this material to be carried along the south shore for a distance of 100 miles, and has built up a nearly continuous beach along that shore.

The foregoing description of littoral drift applies to a continuous beach, and attributes the phenomenon of drift to wave action and undertow. It should be noted that any break in the continuity of the beach eliminates these forces. At such breaks, normally, there are tidal currents of considerable strength to be dealt with. Wave action keeps the bottom more or less agitated, and thereby assists the tidal currents in moving the sands.

It would seem that the question whether littorally drifting sand will be carried across an inlet to the leeward beach, to deep water outside, or into the harbor, is largely a matter of accident. In any event, inlets must result in a waste of beach-forming material, for there are strong tidal currents there which are capable of carrying the material far away from either the windward or the leeward beach.

Jetties and Groins.

As used by the writer, the term "jetty" will refer to structures reaching well out into deep water; the term "groin" will be applied to structures reaching only slightly beyond the low-water line. Wherever there is littoral drift, and jetties have been constructed, beaches have accumulated on the side from which the drift comes. Such is not the case with groins.

Referring to Fig. 7, it is noted that the littoral drift takes place largely below the still-water level. Above that level the sand move-

ment is principally up and down the slope, with only a small component parallel to the shore line. A jetty cuts off the littoral drift, both above and below the still-water level. A groin has little effect on drift due to undertow, and, so far as the drift above still-water level is concerned, a groin is effective for a limited distance only.

One effect of groins deserves serious consideration. Consider a groin so high that no water will over-top it. Such a structure (Fig. 8) will inevitably concentrate both the up-rush and the back-wash of the wave. With a low groin a lesser concentration is effected. It has been the writer's observation that, in the normal case, the berm will be cut back, and a valley will be formed in the vicinity of a groin.

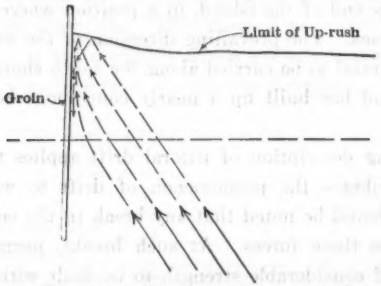

FIG. 8.—A GROIN INEVITABLY CONCENTRATES THE UP-RUSH AND THE BACK-WASH, AND DRIVES THE LITTORALLY DRIFTING SAND INTO DEEPER WATER THAN WOULD BE THE CASE ON AN UNOBSTRUCTED BEACH.

In the vicinity of an inlet, a training dike may be of great value in aiding the formation of a beach, even though it does not reach below the low-water line. At points where channels parallel to the beach (sometimes called swills) are formed, temporary dams across such channels may be of benefit, but, as the channels are transitory, permanent dams are unnecessary.

In order that littorally drifting material may pass a jetty, or a groin, it is necessary for it to travel into deeper water than would be the case on an unobstructed beach, and this deflection reduces the probability of its ever being returned to the beach. For these reasons jetties and groins must inevitably cause a wastage of beach material.

BULKHEADS.

When an engineer is required to build a bridge over a torrential river, he first searches for marks indicating the highest points reached by floods. Using the information thus obtained, he builds to resist

the forces that must be encountered. Similarly, when erecting structures along a beach, he should take note of the signs showing what part of the beach is transitory and what part is fairly stable. The level of the cocoanut grove at San Juan, Porto Rico, and of the sand hills at Long Beach, Long Island, previously mentioned, represent formations that have remained in their present location for many years, and, in the ordinary run of events, may be expected to last for many years. Who builds in front of these stable formations should build strong.

Confronted by a wide berm, such as that at Long Beach, Long Island, the property owner is tempted to erect some form of bulkhead in advance of the line of dunes. The strength of the bulkhead is proportioned to resist the final attack of the waves after most of their force has been spent in crossing the berm. An unfortunate combination of seasons may entirely remove the protecting berm, as it has been removed in former years, and the bulkhead will then be subjected to forces for which it was not proportioned. The result is that the sea takes back its own.

The effect on the berm and foreshore of such substantial sea-walls as are represented by the rip-rap walls guarding the Edgemere Club, the Manhattan Beach Estates, and the neck of land connecting Sandy Hook with the mainland, and of such substanital bulkheads as guard the tracks of the Central Railroad of New Jersey, from Highland Beach to Seabright, should be considered. It is often stated that such structures cause an erosion of the beaches in front of them. The writer believes that this erosion is much less than is commonly thought. The destruction of the berm is usually brought about by the undermining of the outer faces, as indicated in Fig. 6 (*a*), or by a rearrangement of the berm material, to obtain a higher berm of less width, as indicated in Fig. 6 (*c*). In the former case the quantity of material above mean low water is diminished; in the latter case it is increased. In both cases the width of berm is diminished.

Under certain conditions, the back-wash from a sea-wall may tend to undermine its toe by scouring out the bottom for a few feet in front of the wall, but the writer does not recall seeing a single instance of this kind along the many miles of bulkheads near New York City. To erode a berm like that at Long Beach, forces must work far to seaward and must carry the beach material to

deep water, or must, by littoral drift, take it to another part of the beach. A sea-wall would have no material effect on such forces.

The wall in front of the Manhattan Beach property has stopped the erosion of that property, and there is no evidence that it has caused a deepening of the water in front of the wall. There is at the present time a large shoal between Rockaway Inlet and the Manhattan Beach Estates. That shoal is moving steadily westward, and there is no reason to believe that the existence of the wall would prevent the shoal some day from making a contact with the Manhattan Beach property and forming a beach in front of the wall, should other conditions be favorable to such a movement.

The records show that the beach in front of a considerable portion of the sea-wall at the south end of Sandy Hook builds out and cuts away. The sea-wall limits the extent of the erosion, but does not prevent the formation of a new beach when conditions are favorable.

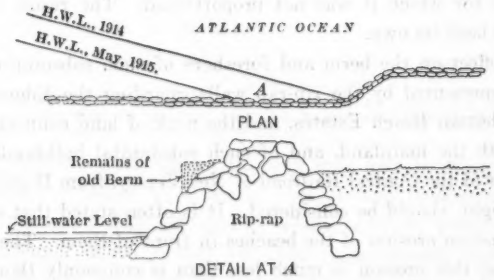

FIG. 9.—AT SANDY HOOK, N. J., THE REMAINS OF AN OLD BERM WERE SEEN WELL UP ON THE FACE OF THE SEA-WALL; BACK-WASH FROM THE SEA-WALL COULD NOT HAVE CAUSED THE EROSION UNDER WAY AT THE TIME.

In connection with this Sandy Hook wall, the writer on one occasion noted casually a condition that should have been carefully recorded and photographed. Its significance, however, was not realized until later. From memory, the conditions were about as shown in Fig. 9. The beach had eroded perhaps 200 ft. in less than a year. The inspection was made in May, 1915, during calm weather. Well up on the wall were the remains of the old berm, just as they had been left by the caving away of the outer part. The corners were sharp, and the caving may have taken place only a few hours before the inspection. It is highly improbable that the sand scarp would

have stood as it was for more than 2 or 3 days, yet the beach had already receded many feet from this particular deposit. The essential facts are that the beach in question was being eroded at a rapid rate by waves that were too small even to splash the old berm. Certainly, back-wash from the wall was not the cause of the erosion, nor was the erosion due to the action of storm waves.

A sea-wall having strength commensurate with its exposure will effectively protect the property in its rear. The location of such a wall, however, should be a matter for serious consideration. If placed so as to include permanently a portion of the temporary berms, the sand thus confined must be deducted from the supply that might otherwise be counted on for the maintenance of the beaches to leeward. The right of a property owner to take permanent possession of beach material temporarily deposited in front of his property may well be questioned. A fair compromise would seem to be that the improvement of the semi-permanent formations be encouraged, and their protection by any means not interfering with the littoral drift be permitted, but that no structures having in view the fixation of the materials composing the temporary berms should be authorized unless the work is on such a scale as to constitute a public improvement, taking care of the beach as a whole.

If used to protect an isolated portion of the water-front, a bulkhead should be placed at about the line separating the temporary berms from the semi-permanent dunes, or in rear of that line. If the protection of the beach as a whole is undertaken, the bulkhead, sea-wall, or detached breakwater may be placed so as to include the temporary as well as the more permanent formations.

It should be understood that the construction of a detached breakwater or a sea-wall enclosing the berms will effectually stop the littoral drift along that frontage until a new beach has been formed outside of the sea-wall or breakwater. If there are beaches to the leeward the effect on them may be disastrous.

Detached breakwaters should afford protection to the beaches in front of which they have been placed. They should create along those beaches conditions similar to those existing in protected bays and harbors. So far as the protected beaches are concerned, rapid changes due to wave action will be done away with, and the beaches may be extended by dredging and filling if desirable.

Conclusion.

It would appear that along each mile of open beach there is a constant transfer of sand from the shore to deep water. When littoral drift is the source from which the supply necessary to maintain the beach is obtained, the quantity of material brought to the section must, in order to maintain the beach, be at least equal to the material carried into deep water plus that carried away by littoral drift.

Assuming a section of beach where the littoral drift is exactly sufficient to replace the wastage, no checking of the sand at any point will benefit the beach as a whole. The valuable part of water-front property is the part corresponding to the level of the cocoanut grove in Fig. 5, but the value of this portion is somewhat dependent on the existence of a sufficient berm at a lower level. Uplands may be protected by sea-walls and bulkheads, but bathing beaches must necessarily lie outside of such structures.

The writer knows of no means by which exposed sandy beaches for surf bathing may be preserved, except by feeding fresh beach material to them as rapidly as the old material is carried away. The rate of growth of Rockaway Point and Long Beach would indicate that many millions of cubic yards per year are required by the beaches of the south shore of Long Island to counterbalance the demands of the littoral drift. There are no data, so far as the writer is aware, by which the quantity of sand carried annually into deep water can be approximated. At all events, it is safe to say that, if the natural supply of beach maintenance material was to be cut off, any attempt to make up the deficiency by dredging or other artificial means would be a stupendous undertaking.

Along the New Jersey shore north of Asbury Park there remain to-day only a few isolated temporary berms. Along most of that shore the waves are eating into the semi-permanent formations or are battering against the bulkheads. It would seem that the only salvation for many sections of that beach lies in the construction of sea-walls of sufficient strength to combat the waves until such time as new berms are formed, if such time ever comes.

A study of the littoral drift on the south shore of Long Island and the disastrous results to certain beaches that have followed tem-

porary stoppages of that drift would be well worth while. The stoppages in the past have been brought about by natural causes, but the works of Man will play an important part in the future, and we may well take warning while there is yet time and see to it that the interests of the millions who annually visit the beaches of the south shore are properly safeguarded.

DISCUSSION

Mr. Haupt.

LEWIS M. HAUPT,* M. AM. SOC. C. E. (by letter).—This interesting paper is limited to the vicinity of the New York entrance, but the principles involved are of general application, with local modifications, so that the writer ventures to call attention to certain features which seem to him somewhat obscure, in the hope of leading to a more definite application of the principles to general practice.

The literature of the subject is prolix, and the publications of the Society, the Institution of Civil Engineers of Great Britain, and numerous other technical societies, contain elaborate discussions of both theory and practice; yet there seems to be room for further analysis, at least of the application of the theory to definite localities, as the results desired are not always attained.

Omitting, for the sake of brevity, the theoretical considerations, as being "somewhat idealized" as to the paths described by the forces which transport the drift, it will be more important to consider the facts as recorded by Nature and, from them, mould our conclusions.

The instrumentalities available for reclamation are jetties and groins, bulkheads and sea-walls "properly applied". This is the crux of the matter, for, in many instances, bulkheads and jetties have proved more injurious than beneficial, and as the local conditions are subject to frequent changes, due to occult cosmic forces, it is very difficult to adjust a permanent structure so as to give satisfactory results under the composite conditions, which is all that the engineer can do.

As to "jetties and groins", a distinction is made, based on their relative lengths, but it would seem better to limit the term to the word "jetty", as a projection from the shore across the strand, than to "groin or groyne", the angle formed by the intersection of two surfaces.

The writer believes that the statement as to their effects, namely, that they "must inevitably cause a wastage of beach material", should be taken *cum grano salis,* for, in the locality in question, short jetties have had a very decided effect in building up the beach without apparent injury to the leeward properties; for instance, at Far Rockaway two jetties (Fig. 10) were erected, about 4 years ago, concerning which the parties say: "The work is highly gratifying as we have gained at least 500 ft. in the width of beach and raised the crest of it away above the top of the piles forming the jetties." The writer is informed, also, that similar jetties built at Long Beach are likewise buried out of sight.

The extensive works now under contract to protect the tracks of the Central Railroad of New Jersey at Seabright, consisting of some thirty jetties reaching out 200 ft., have collected some of the drift at the southerly end, but have not recovered the original low-water line, which was more than 600 ft. out a few years ago. These jetties sup-

* Cynwyd, Pa.

FIG. 10.—HOOKED JETTIES NOW BURIED UNDER THE DEPOSITS WHICH THEY HAVE MADE.

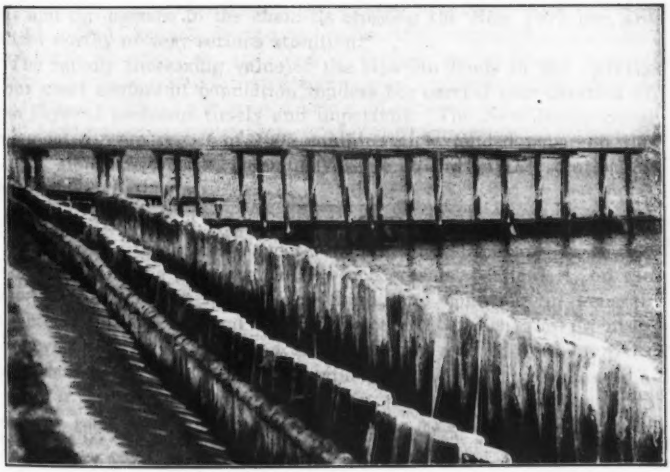

FIG. 11.—REINFORCED CONCRETE JETTY, LONG BRANCH, N. J., FEBRUARY, 1916. BULKHEAD DENUDED AND BACK-FILLING GONE.

plement a heavy bulkhead filled with stone, and this, in severe storms, is scaled by the breakers, which fall over on the track and destroy it. Mr. Haupt.

The influence of the prevailing winds must also be carefully considered, for it is a factor in the accumulation of material, independently of the drift carried by waves and currents. The low, short jetties at Belmar, N. J., during the winter of 1915-16, were completely covered by this wind-driven sand, and under normal conditions they do arrest the northwardly moving drift in the southerly groins, filling them to the crests and traveling over.

Many of the bulkheads along the New Jersey coast have been denuded entirely of their back-filling, and have been further protected by lines of stockades filled in with stone, yet they have not recovered the lost ground, nor do they protect the properties in their rear from the violence of the waves, as has been exemplified so fully by the storms of the past few years, so that it would seem to be unwise to place much stress on this form and location of defensive works. Fig. 11 shows a reinforced concrete jetty at Long Branch, N. J., in February, 1916; the bulkhead was denuded and the back-filling gone.

The statement that "the only salvation for * * * that beach [Asbury Park] lies in the construction of sea-walls of sufficient strength to combat the waves until such time as new berms are formed, if such time ever comes", would seem to be somewhat discouraging, were it not for the fact that, up to this time, no dependence has been placed on "sea-walls" but only on short spur-jetties.

The causes of the temporary stoppages of the drift along the south shore of Long Island, as well as the quantitative movements of the drift and the menace to the channels crossing the New York bar, are matters worthy of very serious attention.*

The rapidly increasing value of the riparian lands in the vicinity of our great centers of population renders the careful consideration of these physical problems timely and important. The New Jersey State Geological Survey report of 1905 contains an extended paper on the changes along that coast which may serve to aid in the solution of these complex problems.

CHARLES H. HIGGINS,† M. AM. SOC. C. E. (by letter).—This paper is most interesting, and the writer congratulates the author on his lucid analysis of the phenomenon of beach erosion, particularly on the coasts of New Jersey and Long Island, and confirms, in general, his observations on littoral drift. However, between the premises laid down as to littoral drift and the conclusion, reached by the author, that: "damage * * * must inevitably result to the beaches as a Mr. Higgins.

* Some allusions to these matters may be found in the *Journal* of the Franklin Institute for February, 1905, and in the *Proceedings* of the Brooklyn Engineers' Club in a paper on the "Reclamation of Coney Island."

† New York City.

Mr. Higgins.

whole if the erection of structures that interfere with littoral drift is allowed to continue", there is a wide gap, which it does not appear to the writer has been logically crossed, and which is certainly not in accord with his observations.

The writer has described* the theory and construction of a system of groins or jetties, constructed at Asbury Park, N. J., under his supervision, 9 years ago, and the results obtained.

Fig. 7 illustrates thoroughly the typical action of waves on a sand beach. In this problem of erosion on a sandy coast, though the masses dealt with are tremendous, the unit is very small, namely, a grain of sand; and, just as the unit of a great army is a single man, and the movements of that army, as a whole, are dependent on the behavior of each of the units, so, if the grain of sand is controlled, by extension the mass may be controlled.

The phenomenon of littoral drift apparently results when the grains of sand are not moved by the waves directly fore and aft, but are sawed along, as indicated by Fig. 7. B_2 is the critical point, for when the line containing the series of points, B_2, moves toward the shore, the observer at once remarks that we are losing beach. On the other hand, when the line moves seaward, beach is made.

Now, the purpose of a system of groins may be single or twofold; that is to say, it may be designed to prevent this critical line, marked "Plunge Point" on Fig. 7, from moving shoreward, in which case the groins end on this particular line; or it may be intended, also, to force this critical line farther out, and thus increase the width of the beach, in which case the ends of the groins extend beyond B_2, thus forcing the shore current farther out, and consequently making beach. An important consideration in such a design is the distance between the several groins. By Fig. 7 the author has indicated clearly the typical path of a grain of sand by lines which form the figure of a saw tooth, B-B_1-B_2. A low groin to the left of B and another to the right of B_2 would, by interrupting this movement, prevent this sawing away of the beach.

The critical line, which the author has named the "plunge point", is really the fighting line between land and sea—the first-line trench, as it were—and as this advances or recedes, a gain is made by one or the other of the contending forces.

Considering the shore as a whole, it is hard to conceive what damage can be expected to result from fixing this "plunge point" line, either where it may be in the particular year the groin system is built, or a few feet or even a few hundred feet farther out. The phenomenon of littoral drift proceeds uninterruptedly on the new line. The author, on the other hand, seems to prefer a bulkhead constructed apparently

* *Engineering News*, April 16th, 1914.

along the line on Fig. 7 marked "Upper Limit Reached by the Uprush". Where bulkheads have been built without a groin system in front, it has usually resulted in the "plunge point" line moving back to the toe of the bulkhead; in other words, the beach is lost; the shore, however, is maintained as long as the bulkhead remains intact. Mr. Higgins.

Now, the only apparent difference in effect on littoral drift is whether that drift occurs along a line through B_1 or along a line somewhat in advance of B_2. The only difference in material, added or subtracted, is the beach between the two lines; and a matter of a few hundred feet, in dealing with an adjustment in masses like sea and land, contains such a comparatively minute quantity of material as to be negligible in considering the total volume.

In very many cases, the protection of the shore, without maintaining a beach, is not sufficient; for example, at Asbury Park, with its mile of ocean front, it is of the greatest importance that a broad sand beach be maintained, and, in general, a shore line with a beach in front of it is preferable to one in which simply a bulkhead marks the limit between land and water. It should be borne in mind that such coasts are developed or used for summer colonies. There are also economic elements entering into this problem. A bulkhead to resist the direct and unimpeded action of the ocean is no trifling affair.

It is the writer's opinion that, in most cases, where shore front for residence purposes is involved, economy, utility, and attractiveness are all best served by a system of construction designed to hold what the author names the "plunge point" line at some distance in front of the shore line—that is to say, dry land—and that this may be accomplished by a properly designed and constructed system of groins or jetties, with a light bulkhead in the rear to prevent wash, the bulkhead being protected from the direct attack of the sea by the beach, which in turn is preserved from erosion by the system of jetties or groins.

It is noteworthy that the author limits his discussion of the New Jersey Coast to that part of it above Asbury Park. The writer is of the opinion that, if the coast above Ocean Grove and Asbury Park received a treatment similar in principle, a similar satisfactory result would be obtained and at a minimum expense.

A massive bulkhead seems to appeal to the imagination, just as a masonry fort does; but, where the impressive fort fails, the simple trenches prove effective. Thus, in preventing the advance of the sea, the low groins hold the line under conditions where the massive bulkhead is smashed to bits.

ALLEN HOAR,* JUN. AM. SOC. C. E. (by letter).—The author of this valuable paper is to be commended on the manner in which he has brought this serious and often difficult problem of beach control to Mr. Hoar.

* Pasadena, Cal.

Mr. Hoar. the attention of the Profession. The paper sets forth clearly and concisely many of the difficulties to be encountered in work of this kind, and points out the necessary investigations to be made before such work can be planned with any certainty of ultimate success. It should be a warning to those who would undertake the planning and building of beach structures without first making careful and thorough investigations extending over a sufficient period of time, and taking into consideration all probable weather conditions. This, unhappily, has not always been done, and many unnecessary failures have resulted.

The writer has been fortunate in being able to make a close study of the beach conditions at Long Beach, Cal., for the past 5 years, and has watched with interest the encroachment of property owners on the waters of the Pacific Ocean along that section known as the West Beach. In this instance, the beach adjoining property has become valuable as a desirable location for hotel-apartment houses and other tourist accommodations, but, as this particular piece of property is just a narrow strip facing the Pacific Ocean on the south and backed up on the land side by high bluffs and the Pacific Electric Railway yards, offering no suitable approach from that direction, it was necessary to build an approach from the east along the beach front. This was also deemed desirable for use as a promenade. For this purpose, and to add to the attractiveness of the beach front property, it was decided to build a wide concrete walk and bulkhead.

The error in judgment here and the difficulties to be met in carrying out the plan adopted were due to the fact that, for the greater part of its length, the southerly line of this property was only a few feet back from the ordinary high-water line, and the walk as constructed, from the property line south, encroached to a considerable extent below the high-water mark and still-water level. The consequence of this has been that, during extreme high water of spring tides, whether accompanied by heavy storms or not, great damage is done to both the bulkhead and walk, and the beach lying in front of the bulkhead is destroyed. Each year since its construction some portion of this bulkhead and walk has been torn out and has had to be rebuilt, and, in fact, it has never been possible to finish the construction at the western end, this section of the work having been destroyed before completion time and again at each attempt.

The bulkhead at and supporting the outer edge of the walk consists of a single row of reinforced concrete sheet-piles, about 6 by 12 in. in section, and from 14 to 16 ft. in length, cast tongue-and-groove fashion. The reinforcement consisted of four vertical rods, one in each corner of the pile, and projecting several feet above the head of the pile for anchorage into the parapet wall to be built after the piles had been placed. The piles were raised by a light pile-driver mounted on skids, and jetted and driven through the sand to penetrate

a layer of clay at a varying depth of from 12 to 14 ft. below the surface. After the piles had been driven to place, a concrete cap was cast in place along the top of the row of piling, and the bulkhead was then back-filled to the level of the cap. The walk, of 6-in. concrete, was then laid, the outer edge being supported on the cap and the remainder resting on the back-fill. Along the outer edge of the walk and over the row of piling a reinforced concrete parapet wall was next cast in place. This wall was built in panels and reinforced horizontally by rods running through its base and through its coping. It was reinforced vertically by the rods which were left projecting above the tops of the piling, serving to anchor it firmly to its base. This construction would have been adequate under all ordinary circumstances, if the bulkhead had been placed at or back of the high-water line, but, placed where it is, in from 1 to 2 ft. of water at ordinary high tides, it is subject to severe impact which becomes terrific with the force of the larger breakers accompanying stormy weather. Mr. Hoar.

The construction of this bulkhead has had a very marked effect on the destruction of beach in front of it. This is undoubtedly because the bulkhead is only a few feet shoreward of the plunge point of the incoming breakers, thereby limiting the extent of the up-rush and its sand-carrying power to a point where the power of the back-flow is predominant; and, in the case of unusually high water, when the plunge takes place almost on the bulkhead, the force and action of the plunge is felt clear down to the sand, and results in excessive scour and erosion. This is due to an almost entire absence of back-flow at this point, and, therefore, instead of spiraling over a comparatively strong back-flow, as is usual, the plunge is carried right to the bottom.

The conditions here are peculiar, in that there is practically no littoral drift of sand or other beach-building material, and therefore no method can be advanced for the rebuilding of the beach by bringing the natural forces into play.

The sand composing this beach is very fine, and contains a very large percentage of extremely light material. The surface of the sand along the beach slopes normally about 2 ft. per 100 ft. to a point about 300 ft. from the high-water line, and from this point it shelves off gradually until it strikes an almost vertical bank about 600 ft. off shore, where the water has an average depth of about 35 ft. There exists just beyond this bank an off-shore current flowing westward, although with no very great force except in case of storm.

The light character of the beach sand allows it to be set in motion by even slight agitation of the breakers, and, when acted on by the large swells raised by a storm, this light material is carried away out by the undertow until it reaches the off-shore current, by which it is carried out to deep water. During calm and low tide much of this

Mr. Hoar. sand is carried only to the outer breaker line where it is again picked up, carried shoreward and deposited on the beach by the up-rush. This is shown by the fact that during calm weather, when the breakers are small, the beach builds up very flat, and, under the influence of the large swells after a storm, it is rapidly cut away between the high- and low-water lines, leaving a comparatively steep slope.

The tidal drift along this beach is from east to west, but the inshore drift is so slow that it is not of the slightest value in transporting sand to build it up.

From the foregoing it can be understood that, in this case, the large sand-laden breakers accompanying heavy storms cannot, because of the location of the bulkhead, aid in the preservation of the beach by building up a new berm, as is the general result as pointed out by Mr. Dent. On the contrary, erosion is accentuated, and a considerable portion of the sand put in motion is transported by the heavy undertow to the off-shore current, where it is lost to the beach for all time.

Mr. Schwiers. F. WILLIAM SCHWIERS,* ASSOC. M. AM. SOC. C. E.—The speaker has had some experience on this subject on the south Long Island shore, and also on the New Jersey shore; and has read considerable on the topics referred to. The Library of the Society contains some volumes written by Mr. Case or Mr. Allanson-Winn, describing the Case system of low groynes. The speaker believes that this system—where its use is feasible—is the best method of shore protection. Briefly, it consists of low barriers, built on the beach, and started below the low-water line. These barriers catch the drift carried by the along-shore currents and build up the beach. As the latter is built up, the groynes are continued shoreward on the proper incline, which corresponds to an elliptical curve, and has been termed the ellipse of repose. The waves ascend the inclined surface until they have spent their energy, and then recede harmlessly. During storms the low groynes are entirely submerged, and are protected by the overlying water.

The Case system has been used extensively in England, but is not adapted to the Jersey shore, because the property owners there have encroached too far on the ocean. In some cases bulkheads have been built even beyond the low-water line, so that no beach is visible at low tide.

The method of protection used along the Jersey shores is timber and pile bulkheads, parallel with the beach, behind which beach sand has been filled in, and low jetties running seaward. The jetties, generally speaking, are low structures entirely submerged during heavy weather, their purpose being to build up the beach in a manner similar to the Case groynes. The destruction of bulkheads takes place during

* New York City.

very high tides accompanied by storms from certain directions. On the south Long Island shore erosion takes place during northeast storms and on the Jersey coast during storms from the southeast. The heavy waves, striking the bulkhead, are deflected upward and downward, creating a semi-fluid mass of the beach material at the foot of the bulkhead. If the sheet-piling has not sufficient penetration, the material back of the bulkhead soon becomes a semi-fluid mass and escapes under the sheeting; this leaves the bulkhead without support, and its destruction rapidly ensues. To support the bulkhead against this contingency, during late years, a second row of brace-piling has been driven about 20 ft. back of the bulkhead, to which horizontal brace logs are attached and connected to the bulkhead piling several feet from the top. This method has been fairly successful, provided there has been sufficient penetration for both the sheet-piling and the round piling. There is sufficient elasticity in this construction, in connection with the material back of the bulkhead, to absorb the shock of the waves. Heavy masonry bulkheads have not been successful along the Jersey shores, and they are not advisable unless an absolutely firm foundation can be obtained and the footing of the wall can be protected from scour by sheeting with deep penetration. Mr. Schwiers.

The situation on the Jersey and Long Island shores is probably aggravated by littoral currents created by the flow of the Hudson. This river, flowing into the ocean, creates shore currents flowing northward along the Jersey coast and westward along the Long Island coast, eroding the southern Jersey shore and eastern Long Island shore, and carrying the material northward and westward, respectively, thus accounting for the formation of Sandy Hook and Coney Island. These littoral currents are reinforced by storms from certain directions, *viz.*: southeast on the Jersey shore and northeast on the Long Island shore; and when both these conditions are coincident with an abnormally high tide, great erosion and destruction ensue.

A. W. Buel,* M. Am. Soc. C. E.—In his discussion, Mr. Schwiers has referred to the Case system of groynes which has been used with great success in several places in England, on both the south and east coasts. These groynes are very low, sometimes only a few inches above the beach, and never more than 18 in. They are arranged so that, as the beach is built up by catching the littoral drift, additional planks can be put in, the groynes thus being raised from season to season. There are records of some places on the English coast where the beach has been built up as much as 8 ft. in a few seasons, and extended out several hundred feet. Mr. Buel.

* New York City.

Mr. Buel. Mr. Case almost invariably had a very low wall at the top of his groynes, that is, at the shore line above high water, and, generally, he used a pavement connecting the beach or the groynes with the sea-wall which he made elliptical. He adopted the elliptical curve as that most nearly agreeing with the angle of repose on the beach under the conditions that exist in those parts of England.

Mr. Allanson-Winn has written several papers* in which he describes the results obtained with Case groynes, and suggests an addition or improvement to the system, consisting of what amounts to extensions of the groynes to a connection with the sea-wall or bulkhead. This extension, from high-water line to bulkhead, is curved in plan. On all the beaches along the New Jersey coast, and also on Long Island, the jetties or groynes are what would be described as high groynes, when compared with the Case system. They are far more substantial and expensive structures than those described by Mr. Case and Mr. Allanson-Winn and, for this reason, are probably less efficient. The very low Case groyne builds up a smoother, more uniform beach than the jetties used on the Jersey and Long Island shores.

The Case groynes were built in England at a cost of about 10 shillings per lin. ft., and Mr. Allanson-Winn's curved extension, connecting the groynes with the bulkhead wall, does not cost more than £1 per lin. ft.

Mr. Cresson. B. F. Cresson, Jr.,† M. Am. Soc. C. E. (by letter).—The question of beach protection raised in this paper is one of great importance to the State of New Jersey, and for several years the condition of the beaches has been under observation by the New Jersey Harbor Commission and, its successor, the New Jersey State Board of Commerce and Navigation.

Particularly was this matter earnestly studied during the winter of 1914-15, when great damage was done to the northerly New Jersey coast between Long Branch and Sandy Hook. The damage was especially great at Seabright, but serious erosions occurred along the Southern New Jersey coast, notably at Longport and Stone Harbor. Plans were prepared by the New Jersey Harbor Commission in 1915 indicating a form of coast protection and urging a comprehensive plan to be adopted in order that the beach might be saved and built, instead of being damaged by the various types of coast protection which have been placed without reference to each other and to a general plan. Each individual property owner usually had his own plan prepared and his own structures built without respect to his neighbor, and the damages that were done were due, at least partly, to the lack of a general study and a general plan. This is notably the case in the vicinity of Seabright.

* *Minutes of Proceedings*, Inst. C. E.

† Jersey City, N. J.

Mr. Cresson.

The writer has read this paper with much interest and considers it a valuable contribution to the somewhat meager literature on this subject, but is compelled to take exception to some of the author's conclusions.

The opening paragraph of the paper states: "The object of this paper is * * * to emphasize the damage that must inevitably result to the beaches as a whole if the erection of structures that interfere with littoral drift is allowed to continue", and, supporting this statement, the author says:

"In order that littorally drifting material may pass a jetty, or a groin, it is necessary for it to travel into deeper water than would be the case on an unobstructed beach, and this deflection reduces the probability of its ever being returned to the beach. For these reasons jetties and groins must inevitably cause a wastage of beach material."

Although this may be the case with high jetties or groins, the writer has observed that it does not apply with low groins. Where low groins are used, and where there is a drift of the sand, the tendency is to build up a beach and numerous instances show it. Groins, 1 or 2 ft. above the level of the sand, will allow the current to pass over them, and, in doing so, retard the speed of the drift; then the sand in suspension will be deposited on both sides of the groin. There are numerous instances to show that this is the case.

The groins which the Central Railroad Company of New Jersey has recently built in the vicinity of Monmouth Beach have already commenced to gather sand, and, in conjunction with a sloping bulkhead, appear to be the most effective method of reclaiming the beach and stopping erosion.

The writer can scarcely agree with the following conclusion by the author:

"It would seem that the only salvation for many sections of that beach lies in the construction of sea-walls of sufficient strength to combat the waves until such time as new berms are formed, if such time ever comes."

The writer believes that a combination of low groins and a bulkhead or sea-wall with a sloping, curved, or broken face, so that the receding waves may not scour at the toe of the wall and carry the sand to sea by the undertow, is the most effective method of gathering beach and of protecting the foreshore. Neither groins alone, nor bulkheads alone, will accomplish this result, in the writer's judgment, under conditions such as exist along the New Jersey coast.

Instead of attempting to combat the waves by the strength of a heavy sea-wall, the writer believes it is far better to attempt to guide the forces of Nature by permitting the sand to collect in front of the bulkhead by the placing of low groins; the sand itself will then form a protection for the bulkhead.

Mr. Dent.

ELLIOTT J. DENT,* M. AM. SOC. C. E. (by letter).—In the first paragraph of the paper, its application was limited to that portion of the New Jersey shore north of Asbury Park. Asbury Park should have been included, as that beach was visited on several occasions, and the remarks and conclusions are applicable to it.

The discussion has been directed principally toward the writer's remarks in regard to groins and the detrimental results that must inevitably result if structures interfering with the littoral drift are erected along the beaches under consideration. Before taking up the discussion in detail, a few additional general remarks in regard to littoral drift are necessary.

Littoral drift is generally spoken of as though it were of constant magnitude; beaches are referred to as being in equilibrium, due to the fact that the littoral drift brings to them as much material as it takes away. As a matter of fact, no such equilibrium will be found to exist along beaches around New York City; such beaches undergo erosion for a period of years, and subsequently there is a period of accretion; the growth and denudation of the beach occur in what may be styled "cycles". The cause of these cycles is apparently the progress of sand waves along the shore. If the point where the temporary berms are widest is called the crest of a sand wave, and the point where the semi-permanent dunes are subjected to attack is called the trough, two such waves were in evidence during the summer of 1915. In one case the trough of such a wave was traveling along the sea wall at the south end of the Sandy Hook Military Reservation, while wide berms, corresponding to the preceding crest, might have been seen a mile or so farther north. At Long Beach, temporary berms of considerable width existed in front of the boardwalk and for a distance of perhaps 2 miles to the east; the berms then disappeared, and, for a considerable distance, the sea was making a direct attack on the semi-permanent dunes; still farther east, in the vicinity of Point Lookout, wide berms were again in evidence.

The material comprising the temporary berms is loose and friable, but when they have been removed, the remaining beach material along the New Jersey shore will be found to contain a large percentage of gravel, which offers considerable resistance to further erosion. Whatever may be the cause of the added resistance, the beaches along the Long Island shore appear to be much harder and much less susceptible to erosion after the temporary berms have been removed than they are while such berms are in existence; the temporary berms are loose under foot, and walking along them is a considerable labor, while, at the same elevation, where the berms have been removed, the beach will be found to be fairly compact and the walking easy.

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* Little Rock, Ark.

Mr. Dent.

Suppose, for example, that an engineer has been called on to design works for the protection of a beach 1 mile in length, and that, due to the passage of the trough of a sand wave, this frontage has been subjected to an attack. Suppose, further, that the crest of a new sand wave is approaching. If no interests other than those of this particular mile of beach are to be considered, it may be feasible to stop the passing of the littoral drift by the erection of a jetty of such height and length that no sand will pass around or over it until the windward beach has grown up to the desired extent. The cove formed on the windward side of this jetty having been once filled by the littoral drift, and the shore line having advanced to the end of the jetty, the drifting sand would proceed around its end, and would eventually travel along the leeward beaches as though no obstruction had been created.

For the protection of such a beach, Mr. Higgins proposes the use of groins. According to him, such groins will cause the line marked "Plunge Point", on Fig. 7, to advance seaward, and the other lines to advance a similar distance. Mr. Higgins states that the quantity of material required to extend the beach "a few hundred feet" is "negligible".

Conceding for the moment that the line marked "Plunge Point" and the other lines of the beach may be moved seaward by the construction of groins, the following propositions will logically follow:

(*a*).—The material used to build up the beach is the same in the case of the jetty as in the case of the groins.

(*b*).—On account of the increased depth of the reclaimed areas, as compared with the depths over the areas that would have been reclaimed by the natural berms, the former area will be less than the latter.

(*c*).—Fixation of the drifting sand to reclaim the artificial beach will postpone for a corresponding period the date when the leeward beaches will receive a new supply of material.

(*d*).—Dependent on the particular location, the length of beach benefited may be less than that injured.

If the drift was passing in a steady stream, the erection of a jetty or of a system of groins (still assuming for the sake of argument that a system of groins will check the littoral drift) would result in the creation of an artificial trough, and the leeward beach would be subjected to the attack which always accompanies the passage of such a trough. Other conditions might be assumed, but the conclusions would be in no wise modified.

Any structure that stops and retains the littorally drifting sand in front of any particular property must inevitably induce an attack on the leeward property. It is an extraordinary fact that though the stoppage of the littoral drift by a jetty is so generally recognized as

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being detrimental to the leeward beaches, it is claimed, nevertheless, by numerous writers, that the stoppage of the same drift in front of the same property may be effected by the use of groins without damaging the leeward property.

Turning now to the questions raised as to whether groins will actually check the littoral drift and cause an accretion to the beach in one case, or whether a system of groins will retard the erosion of an existing beach in another case, it will be well to examine into the results actually visible along the Long Island and New Jersey shores.

The sea-wall at the south end of the Sandy Hook Reservation has been referred to several times. Since the paper was written, it has been learned that, in 1901, six timber groins, of the type usual along the New Jersey shore, were built near the north end of that wall. The writer visited the locality several times during 1914 and the first half of 1915, and the groins were buried so completely that their presence was not suspected. The fact that, after the erection of the groins, the beach was extended seaward until they had been completely buried so that they were no longer able to affect in any manner the growth of the beach, and the fact that the growth of the beach did not stop there, would indicate that the groins were not the primary cause of the accretion. The alignment of the beach would have led to a similar conclusion if the existence of the groins had been known.

In May, 1915, erosion was taking place along the face of this sea-wall, as described on page 1802. The trough of a sand wave was traveling north along the shore. As this trough continued its northerly course, it encountered the first groin of the group. The existence of the groin did not stop the erosion. By November two groins had been exposed for their entire length and a part of a third one was bared. The writer has no information as to what has occurred since 1915, but here is a case where there is strong presumptive evidence that the growth of the beach was due to causes entirely independent of the erection of groins, and there is positive evidence that the groins were impotent, so far as the preservation of the beach was concerned, when conditions had so changed that a period of erosion was due.

Mr. Haupt states that jetties similar to the one shown in Fig. 10, at Long Beach, Long Island, are now "buried out of sight". The writer in 1915 walked the entire length of Long Beach, from Point Lookout on the east to East Rockaway Inlet on the west, and noticed no evidence of their existence. As in the case of the Sandy Hook groins, the fact that the Long Beach structures were completely buried creates a presumption that the growth of the beach was due to some cause other than their construction. Mr. Haupt has described* certain shore protection works at Edgemere, Long Island, and has presented a map showing that shortly after the erection of these structures there

* *Proceedings*, Brooklyn Engineers' Club, 1913.

was a material growth along the corresponding frontage. The same map, however, shows that the growth of the beach, along the frontage which it was aimed to protect, was not measurably greater than that of the beach during the same period for a considerable distance east and west of the protected section. Is this not presumptive evidence that the accretion noted was due to causes entirely independent of the erection of the works designed by Mr. Haupt? Mr. Dent.

The hooked jetty shown in Fig. 10 was by no means buried at the time the photograph was taken. If any accumulation took place after its erection, no evidence has been presented to show that it was due to the presence of the structure. The evidence furnished by this photograph would indicate that there was a considerable depth of water at the outer end of this hooked jetty, and the photograph does not show any materially greater deposits on the windward than on the leeward side.

Mr. Higgins has described as follows, the work that he advocates for the New Jersey shore above Ocean Grove and Asbury Park:

"It is the writer's opinion that, in most cases, where shore front for residence purposes is involved, economy, utility, and attractiveness are all best served by a system of construction designed to hold what the author names the 'plunge point' line at some distance in front of the shore line—that is to say, dry land—and that this may be accomplished by a properly designed and constructed system of groins or jetties, with a light bulkhead in the rear to prevent wash, the bulkhead being protected from the direct attack of the sea by the beach, which in turn is preserved from erosion by the system of jetties or groins.

"* * * The writer is of the opinion that, if the coast above Ocean Grove and Asbury Park received treatment similar in principle, a similar satisfactory result would be obtained and at a minimum expense."

Mr. Cresson favors the same system.

Long Branch is situated along the shore referred to by Mr. Higgins, and, in Fig. 11, Mr. Haupt shows structures of the type recommended, namely, a system of groins backed up by a bulkhead. The illustration shows that the groins have not held the plunge point at a distance from the bulkhead, and that no protecting beach has been built up to prevent the direct attack of the waves on the bulkhead. Fig. 11 is typical of what may be seen at many points along the New Jersey shore. To be sure, Mr. Higgins limits his claims to a "properly designed and constructed system of groins or jetties", and Mr. Cresson limits the height of his groins to 1 or 2 ft. above the surface of the sand, but the average engineer is unable to find any essential difference between the structures now existing along the New Jersey shore and the structures advocated. Some explanation as to why existing structures have failed should be forthcoming before we are asked to place our faith in additional work so nearly like the old

Mr. Dent. that we are unable to see any essential difference. Mr. Haupt calls attention to the fact that, in many instances, "bulkheads and jetties have proved more injurious than beneficial".

At Sea Gate, on the Long Island shore, a system of groins backed up by a light bulkhead proved no more successful than the one at Long Branch illustrated in Fig. 10. As a result of damage done during the winter of 1914-15, an offshore breakwater was started during the summer of 1915 for the better protection of the Sea Gate property. Mr. Haupt has referred to two jetties built at Far Rockaway about 4 years ago. Concerning these jetties it is stated that the work is highly gratifying, as at least 500 ft. in width of beach has been gained, and the crest of the beach has been raised away above the top of the piling forming the jetties. As stated previously, when the growth of the beach is such as to bury out of sight the groins or other protection works, it is, to the writer, presumptive evidence that the cause of this growth is due to some condition entirely independent of the existence of the structures. During the winter of 1914-15, Far Rockaway and Edgemere were severely battered by storms. During the summer of 1915 several wrecked groins in this vicinity were visible, but the only work in sight which appeared to be protecting successfully the property which it was supposed to guard was the rip-rap bulkhead around the Edgemere Club. In 1910, there were at least ten groins or jetties in that vicinity, but, with the evidence before them, the land-owners of Edgemere decided, in the summer of 1915, to construct an off-shore breakwater for the protection of their property. The work was started during the same summer.

The foregoing refers to the general effect of groins, and the writer's conclusion has been that they are totally unable to build up a denuded beach or to prevent the denudation of an existing beach. It remains to examine more closely into the effects visible in the immediate vicinity of the numerous existing groins along the beaches under consideration. At the same time there will be discussed exceptions taken by Messrs. Haupt and Cresson to the writer's statement that, by forcing the littorally drifting sand into deeper water than it would reach under natural conditions, jetties and groins must inevitably cause a wastage of beach material.

It is generally claimed for groins that they stop and hold the littorally drifting sand in place until the windward beach has been built up to their tops. Thereafter, the littoral drift passes over the tops of the groins unimpeded. A few rare examples of this condition may be found along the beaches around New York City, but the normal condition is very different. In the normal case there will be found in the vicinity of the groin a slight depression of the beach from about mid-tide to the crest of the berm, as shown in Fig. 12. Examples of this have been noted at many points along the New Jersey shore, including

Mr. Higgins' groins at Asbury Park, and detailed notes were made during the summer of 1915, in connection with fifteen groins built according to the Case system in front of the property of the Lido Corporation, a short distance east of the end of the boardwalk at Long Beach, Long Island. Mr. Dent.

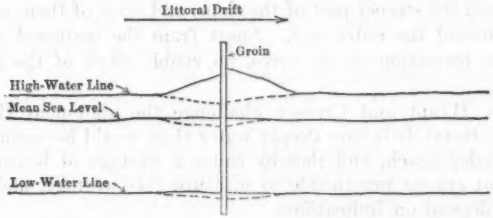

FIG. 12.—SOLID LINES REPRESENT THE NORMAL FORM OF BEACH AS OBSERVED IN THE VICINITY OF GROINS ALONG THE NEW JERSEY AND LONG ISLAND SHORES. THE BROKEN LINES REPRESENT THE FORM THAT SHOULD EXIST IF THE GROINS CAUSED A DEPOSIT.

This depression forms what may be styled a cove. During the early part of the summer, the coves in the vicinity of the Lido Corporation groins were hardly visible; the berm had a rounded appearance, as though it had been recently formed, and its width was about the same along the section covered by the groins as along the beach to the east and west. Throughout the summer, erosion was in progress and, on September 5th, the depths of the various coves were as given in Table 1; the bents are about 8 ft. in length:

TABLE 1.

Groin No.	Depth of cove.	Groin No.	Depth of cove.	Groin No.	Depth of cove.
1	2 bents	6	2 bents	11	2 bents
2	3 bents	7	2 bents	12	2 bents
3	2 bents	8	2 bents	13	2 bents
4	2 bents	9	1 bent	14	2 bents
5	2 bents	10	4 bents	15	2 bents

According to Mr. Cresson, low groins such as these should cause the sand to be deposited on both sides of the groin, in which case the beach should take the form shown by the broken lines on Fig. 12. It will be noted that the writer's statement of observed fact is here the direct opposite of what Mr. Cresson states is shown by numerous instances.

According to groin advocates, the slope of the beach on the windward side should be determined largely by the slope of the groin. Around New York City such is not the case in fact. In the case of the Lido Corporation groins, the beach always showed one of the forms given in Fig. 6; for a variable number of bents on the shoreward end of the groins the planks were completely buried, only the tops of the

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posts showing through the nearly level berm; at the outer edge of the berm a typical scarp indicative of erosion was to be seen during most of the summer; below this scarp the slope of the beach was normal from the upper limit reached by the up-rush to the limit laid bare at low tide. The planks were wholly buried on the shore end, partly buried along the steeper part of the slope, and some of them were under-scoured toward the outer end. Apart from the scalloped appearance due to the formation of the coves, no visible effect of the groins was noted.

Messrs. Haupt and Cresson challenge the statement that groins force the littoral drift into deeper water than would be encountered on an unimpeded beach, and thereby cause a wastage of beach material. It does not appear practicable to measure this wastage, and we must therefore depend on indications.

At high water some sand may be thrown, by the up-rush, over such groins as are shown in Figs. 10 and 11; the back-wash, however, does not lift the sand far from the bottom, and little or no beach material will be carried over a high or low groin during its downward passage. As little material passes over the groin at high water and none at low water, we should have an accumulation of sand on the windward side of the groin if it were not carried either around the end of the groin or out to sea; the absence of this accumulation leads the writer to conclude that the sand is in fact forced into relatively deep water in passing the obstruction.

At Highland Beach, N. J., there was one groin which, throughout the period of observation, was apparently performing its functions in the manner claimed by the groin advocates. On the southerly side of the groin the beach was built up to the top of the sheet-piling, and the slope was the same as that of the groin; on the northerly side the beach was about 3 ft. lower. The only explanation that the writer has been able to suggest for this exceptional condition is that the beach in that locality contained a considerable proportion of gravel. Some writers on the subject of shore protection along the British coasts have stated that either high or low groins readily hold shingle or gravel. They emphasize the fact that retaining littorally drifting sand is a much more difficult problem.

The foregoing discussion of groins should not be taken as a criticism of their use along the British coast. The conditions there are very different from those in the vicinity of New York Harbor. At all the localities in England which the writer has studied, the tides are from 8 to 20 ft. in height; in many of these localities the berm material is shingle or gravel, and the problem of protection is more often the protection of an eroding cliff than the protection of a sandspit, as in the case of the New Jersey and Long Island Beach Colonies. This paper has been given a special and limited application, and no attempt

has been made to show what parts of the discussion are of general, and what are of purely local, application. Mr. Dent.

The remaining points brought out during the discussion will be taken up in order.

Mr. Haupt states that the jetties at Belmar, N. J., arrest the wind-driven sand. It may be noted that along the Rockaway beaches brush fences are used for the same purpose. The writer's discussion was limited to the effect of wave action.

The writer did not claim that bulkheads could recover lost ground, but it was claimed for this type of structure that the semi-permanent dunes could be adequately protected, and that this form of protection would not prevent the formation of bathing beaches or berms in front of the bulkhead when other conditions were favorable.

The drift along the south shore of Long Island is of enormous magnitude, but the writer has not been impressed by the menace of this drift to the entrance channels to New York Harbor. It is believed that if Rockaway Point continues to grow in a westward direction for another 75 years at the rate at which it has grown during the past 75 years, the results will be positively beneficial. The latest soundings in Ambrose Channel do not indicate any serious encroachment of the Long Island sand.

Mr. Higgins has evidently not understood the meaning of the term "plunge point" as used in this paper. The line marked "plunge point" on Fig. 7 moves back and forth with the rise and fall of the tides, and its location is also somewhat dependent on the height of the waves. Along the Long Island shore the distance moved by this line during a single tidal oscillation may be more than 150 ft., and it is manifestly impossible to terminate a groin on a movable line. In a tideless sea, where it might be so terminated, it is not clear how the groin would then hold the line in a fixed position.

The distance from B to B_2 in Fig. 7 is normally less than 50 ft. Such being the case, placing a groin to the left of B and another to the right of B_2 would constitute a closer spacing than the writer has ever seen seriously advocated. It is believed that Mr. Higgins has misunderstood the writer's meaning in this instance also.

There is no objection to fixing the alignment of the beach, either as it exists to-day or at some point farther seaward, provided the material for advancing the beach is not taken from the littorally drifting sand on which the leeward beaches depend for their maintenance, and provided the ends of the improved section are arranged so that they will not interfere with the littoral drift; such rectangular off-sets as occur at Seabright should be avoided. On page 1803 the writer gave his conclusions as to the proper location for a bulkhead, fixing the limit to which the sea might be allowed to encroach.

Mr. Dent. The maintenance of a bathing beach in front of a bulkhead is of great importance, one of the most serious questions with which the beach protection engineer is concerned being how to maintain such a beach. It is believed that a brief examination of the shore lines of Long Island and New Jersey will convince any engineer that the present methods have not proved successful. In many cases the measures adopted have not only failed to maintain a bathing beach, but have also failed to protect highly valuable property built on the semi-permanent dunes.

Mr. Hoar cites a case radically different from that under consideration. Long Beach, Cal., is on a bay, and the beach has in the past been built up and maintained with material found locally, and not with material brought by littoral drift from a distance. Wherever the temporary berms are enclosed within a bulkhead, as appears to have been the case at Long Beach, Cal., those responsible for such a construction should realize that, unless a new supply of material is in sight, their action will destroy the bathing beach, and the bulkhead will be subjected to a direct attack.

Mr. Hoar also misunderstands the term "plunge" as used by the writer. On the Pacific Coast where the prevailing winds blow on-shore, sands temporarily deposited above the low-water line are frequently, as soon as they dry out, blown shoreward, where they form traveling dunes. On such a shore the temporary berms are destroyed by an agency totally different from the one discussed in this paper. On some of the Pacific Coast beaches this wind action is responsible for the very flat slope that will be found between low tide and the upper limit reached by the waves at high tide. On such a beach the effect of the plunge may be much less important than on the beaches around New York City, in fact, its importance may be practically negligible.

It should be emphasized that, as the term is used by the writer for the beaches around New York City, the "plunge" always reaches clear to the bottom and causes a violent disturbance of the sand. Seaward of the plunge point the breaking waves have slight capacity for disturbing or picking up the sand. Mr. Hoar states that during calm weather and low tides much sand is carried to the outer line of breakers "where it is again picked up, carried shoreward and deposited on the beach by the up-rush." On the beaches around New York City, the resultant forces seaward of the plunge point tend to carry the sand to deep water; though the waves are able to disturb the bottom sufficiently to cause a slight turbidity in the water at the outer line of breakers, the quantity of sand brought to the surface seaward of the plunge point is negligible. The phenomena described by Mr. Hoar for Long Beach, Cal., are not duplicated along the beaches under consideration.

Mr. Hoar also refers to the plunge as spiraling over a comparatively strong back-flow. This is a phenomenon described by Messrs. Owens and Case,* as follows: Mr. Dent.

"* * * if the waves strike the shore at such frequent intervals that the back-wash of one is met by the up-rush of the following wave, a very peculiar state of affairs is produced. At first sight, one would say, here is a case where there must be great accumulation going on, since the back-wash is met in this way by the up-rush and its scouring action presumably destroyed; but no, this is a most deceptive appearance, and is not borne out by closer observation, for instead of the checking of the back-wash by the water of the incoming wave, it simply glides up over the surface of the back-wash, thus completely reversing our first conclusion; for here we have an under current flowing seaward, and, on the top of it, a landward current. It is thus obvious that the landward current cannot pick up any material from the bottom, and some of what it may already have in suspension will be robbed from it by the down-flowing under current."

Such conditions may be found occasionally on the beaches under consideration, and, when they exist, it is to be expected that erosion will take place. The writer has often watched for this condition, but has never seen a case of the sort.

Messrs. Schwiers and Buel refer to the so-called ellipse of repose as being the typical form taken by a beach. Such a form will be found along the eroding cliffs of the north shore of Long Island and Gardiner's Bay. To some extent the same form is developed along the beaches under consideration wherever the temporary berms have been washed away and the semi-permanent dunes are subjected to attack. Where a beach is building up, and where a system of Case groins is being raised to meet this upbuilding, the form of the beach and the profile of the top of the groin will be as indicated in Fig. 6.

In stating that on the south shore of Long Island erosion takes place during northeast storms, and that, on the New Jersey coast, the erosion is caused by storms from the southeast, Mr. Schwiers has raised the question of the effect of the direction of the wind. The writer believes that the direction of the wind has little or no influence on the question of erosion or accretion. At the Condado Beach, San Juan, Porto Rico, it can be stated positively that the berms are eroded by waves of small magnitude and built up by waves of greater magnitude; the direction of the wind in this case being unimportant. A very common statement on this subject is that on-shore winds erode a beach, and off-shore winds build it up. Relative to this the records of the Weather Bureau for New York City for the 10 years, 1905 to 1914, are interesting. During that time the wind gauge showed 336 off-shore gales for the New Jersey coast, 41 on-shore gales, and 34 gales parallel to the shore. If off-shore gales cause an accretion, the New Jersey

* "Coast Erosion and Fore-Shore Protection", pp. 15 and 16.

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beaches should not have suffered as they have during the past few years. In direct opposition to Mr. Schwiers' statement, that the New Jersey coast is eroded by storms from the southeast we find the statement of Mr. Higgins:* "it is the northeasters that erode the beach." (Asbury Park, N. J.)

Mr. Schwiers states that heavy masonry bulkheads have not been successful along New Jersey shores. Mr. Haupt says that they have not been tried, and the writer knows of none north of Asbury Park. The rip-rap bulkhead at the south end of Sandy Hook has protected that neck of land successfully, and the substantial pile and rock bulkhead built by the Central Railroad of New Jersey and extending from Highland Beach to Seabright has also been fairly successful. The writer believes that bulkheads of sufficient strength to protect the semi-permanent formations can be built, and that therein lies the most feasible solution of the problem with which the New Jersey beach colonies are confronted. It is a fact that sometimes, as stated by Mr. Higgins, "where the impressive fort fails, the simple trenches prove effective," but it is also a fact that it would be futile to detail a corporal's guard to stop a Prussian division.

In reference to the Case system of groins, Messrs. Schwiers and Buel seem to have been unaware of the fact that fifteen such groins were in existence a short distance from the boardwalk at Long Beach. Speculation based on experiences with structures of this type along the shores of Great Britain would not be profitable in view of the ease with which their actual effect on the beaches around New York may be observed.

The writer agrees with Mr. Cresson in his indictment of the present system, and in the statement that Seabright is a notable illustration of the evil effects of structures built without sufficient regard to a general plan.

The plans submitted by Mr. Cresson for a bulkhead at Longport, N. J.,† show a much more substantial structure than has generally been built in the past. This is a step in the right direction, but the writer cannot concede that the erection of groins in front of such a bulkhead gives promise of building up a protecting sand beach, or of holding an existing sand beach if a period of erosioin is due.

It is a pleasure to note that in New Jersey the subject of beach protection is receiving official notice, and it is hoped that in due course of time reports showing measured changes in the beaches, together with numerous facts in connection with the progress of these changes, may be made public. A comprehensive study, including the recording of observed facts, should explain many of the phenomena that, as Mr. Haupt expresses it, must now be attributed to "occult cosmic forces".

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* *Engineering News*, April 16th, 1914, p. 832.

† *Engineering Record*, May 1st, 1915, p. 547.

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Paper No. 1376

A STUDY OF THE DEPTH OF ANNUAL EVAPORATION FROM LAKE CONCHOS, MEXICO*

By Edwin Duryea, Jr., M. Am. Soc. C. E.

AND

H. L. Haehl, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. T. K. Mathewson, E. F. Chandler, William S. Post, John E. Stirling Thorpe, Charles H. Lee, Robert Follansbee, L. R. Jorgensen, Adolph F. Meyer, H. Hawgood, Charles W. Comstock, C. E. Grunsky, H. A. Whitney, M. Hégly, Robert E. Horton, J. W. Ledoux, and Edwin Duryea, Jr., and H. L. Haehl.

Synopsis.

This paper treats of the determination of the yearly evaporation depth from a large reservoir, in an instance where it was of unusual importance to ascertain a safe and reasonably accurate value, where there were no existing evaporation data for places nearer than 300 miles, and no local data except mean temperatures and elevation above sea level. It treats also of the checking of the depth thus estimated by local evaporation observations started for that purpose.

Lake Conchos is a large artificial reservoir, about 300 miles south of El Paso, Tex., with an area (at spillway level) of 67.7 sq. miles, a storage capacity of 2 550 000 acre-ft., a maximum depth of 223 ft., and an average depth of 61 ft. The lake is about 4 300 ft. above sea level, and the mean yearly temperature of the region is about 67° Fahr.

* Presented at the meeting of November 17th, 1915.

The waters of the lake are to furnish electric power to the Parral mining district, about 50 miles distant; and the project's dams, power-house, transmission line and Parral Sub-station already (May, 1915) are so nearly completed that power will be delivered at Parral during the present summer.

Due to the great lake area and the probable great depth of yearly evaporation, the evaporation losses from this reservoir are of unusual importance as an element affecting the net water supply available for power production; and since widely differing opinions had been given as to the yearly evaporation depth, it was necessary to ascertain for it a safe and reasonably accurate value.

The yearly evaporation depth from the lake was estimated by three methods, *A*, *B*, and *C* (*B* and *C* each having two variants).

Method *A* is based on data of pan evaporations, mean temperatures, and elevations above sea level—all as observed at six stations in Texas and New Mexico, distant from 300 to 500 miles from Lake Conchos. It is also based on the known elevation of the lake, and the temperatures there, as observed for a period of about 2 years.

Method *B* consists of a modification and amendment of the yearly evaporation depth, estimated by Method *A*, by the aid of first 2 months' and later 5 months' observations of pan evaporations at Lake Conchos.

Method *C* consists of the estimation of the yearly evaporation depth from Lake Conchos by means of the excess of the inflow into the lake above the outflow—first for a period of 4 months and finally for a period of 7 months.

The yearly evaporation depths from the lake, estimated by the three methods, are as follows:

Method *A*	.....	52.5 in.
Method *B* (2 months)	B_a = ... 52.9 in.; B_b = ...	52.4 in.
(5 ") "	= ..... 54.1 " ; " = ...	53.4 "
Method *C* (4 months)	C_a = less than 55.4 in.; C_b = less than	56.2 in.
(7 ") "	= " " 57.7 " ; " = " "	59.3 "

At the end of the 2 months' observations, the value of 55 in. was adopted; and, as the final conclusion from the whole investigation, 55 in. was re-adopted, unchanged, as the safe yearly evaporation depth.

Besides the foregoing principal conclusions of this investigation, there are statements of the relative evaporation depths from pans from 2 to 6 ft. square, data and conclusions as to the relative evaporation depths from land-pans and from floating-pans of the same size, and data and conclusions as to the effect of mean temperature and of elevation on the depth of evaporation.

Also (and this constitutes the most important feature of the paper), it is estimated by a combination of the results of Methods *B* and *C* that at Lake Conchos the evaporation depth from the lake surface certainly is less than 67.5% (and in all probability as little as 62%) of that from a pan 3 ft. square floating thereon. In 1910 Professor Frank H. Bigelow, of the United States Weather Bureau (in connection with his Salton Sea evaporation experiments), deduced a value of 62% for this coefficient; and, so far as known to the writers, the foregoing value at Lake Conchos is the only check of Professor Bigelow's value which ever has been made.

General.

Lake Conchos is an artificial reservoir in the State of Chihuahua, Mexico, about 20 miles west of the City of Santa Rosalia, 90 miles south of the City of Chihuahua, and 300 miles south of El Paso, Tex. The lake is on the Continental Plateau to the east of the Western Sierra Madre Range, is at an elevation of about 4 300 ft. above sea level, and is in a region having a mean temperature of about 67° Fahr. Its climate is very similar to that of El Paso, Tex., which is at an elevation of 3 700 ft., and has a mean temperature of about 63° Fahr.

The shape and extent of the lake are shown by the map, Fig. 1; and the general character of the shores and of the surrounding country is shown by the panoramic photograph, Fig. 2.

The lake or reservoir is formed by impounding the waters of the Rio Conchos (a tributary of the Rio Grande) by three dams—the La Boquilla Dam across the Rio Conchos, a masonry structure of 244 ft. maximum height and 840 ft. crest length; the Tigre Hill Dam, an auxiliary dam of 90 ft. maximum height and 2 780 ft. crest length; and the Babizas Spillway Dam, of 31 ft. maximum height and 3 190 ft. crest length. The La Boquilla Dam is a curved masonry structure of heavy gravity section (proportioned for "uplift"); the Tigre Hill

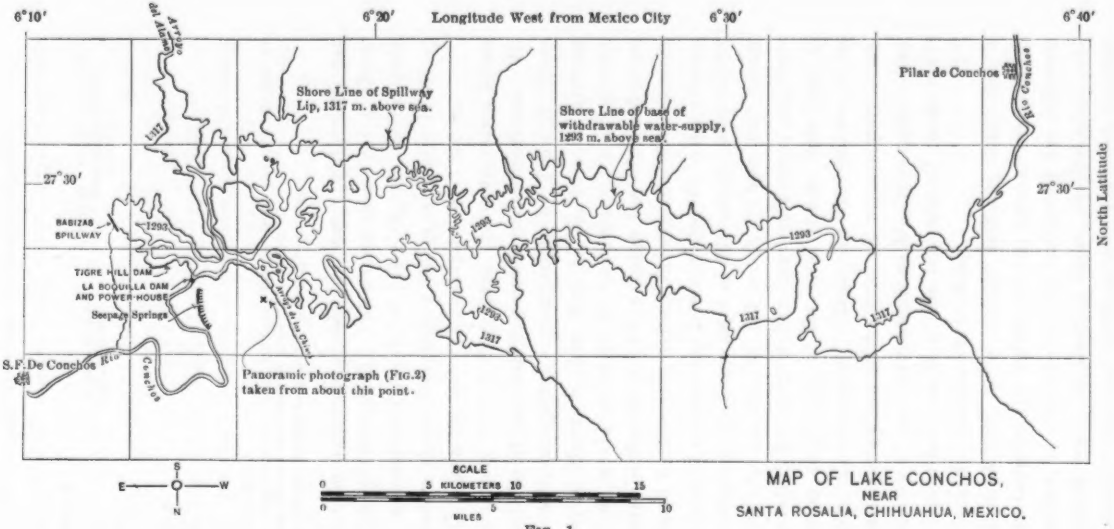
is a masonry dam of inadequate gravity section, strengthened by an earth embankment against its down-stream face; and the Babizas Dam is to consist of a heavy, gravity-section, masonry spillway, 2 360 ft. in length, flanked at its ends by earth embankments. The La Boquilla Dam is of cyclopean rubble laid in Portland-cement concrete. It is now (May, 1915) about 97% completed, and should be entirely completed within a month or two. The Tigre Hill is of hand-laid rubble in hydraulic-lime mortar, and is now nearly completed. The Babizas Dam is to be of hydraulic-lime concrete, and only its foundation excavation has been done as yet. The La Boquilla Dam now is sustaining a maximum depth of water of about 190 ft., and there is a maximum depth of about 15 ft. of water against the Tigre Hill Dam.

The plans for the three dams were made by William B. Fuller, M. Am. Soc. C. E. (formerly Chief Engineer), after general plans made by John R. Freeman, M. Am. Soc. C. E., Mr. Freeman's plans being restricted in some ways because of the necessity of making use of parts of the work previously designed and constructed by Messrs. S. Pearson and Son, the contractors during its early stages. The major part of the work as yet constructed was done by company labor, under the supervision of Mr. Fuller, who was the Chief Engineer of the Mexican Northern Power Company, Limited, from the fall of 1911 until August, 1913. The work is being completed by company labor, under the supervision of the present Chief Engineer, G. G. Underhill, Assoc. M. Am. Soc. C. E.

The purpose of the reservoir and of the project is to supply electric power to the Parral mining district, about 50 miles distant, from a power-house just below the La Boquilla Dam. This power-house, including its machinery, already is practically completed, and also the transmission line and the Parral sub-station; and contracts were made recently to deliver electric power at Parral during the summer of 1915.

Evaporation.

Toward the end of 1913, the writers found it necessary to investigate the evaporation losses from Lake Conchos, as one of the elements of a general investigation of the partly constructed power project. The lake is to be one of the largest artificial reservoirs in the world, its surface areas and storage capacities being as given in Table 1.



MAP OF LAKE CONCHOS, NEAR SANTA ROSALIA, CHIHUAHUA, MEXICO.

FIG. 1.

FIG. 2.—PANORAMIC VIEW LOOKING SOUTH ACROSS LAKE CONCHOS, FROM THE POINT MARKED × ON FIG. 1. FROM A PHOTOGRAPH TAKEN SEPTEMBER 15TH, 1914; WITH LAKE SURFACE AT ELEVATION 1308.5 M. ABOVE SEA LEVEL AND 44.3 FT. BELOW ELEVATION OF SPILLWAY LIP.

TABLE 1.—AREAS AND VOLUMES OF LAKE CONCHOS.

ELEVATION OF LAKE SURFACE ABOVE SEA LEVEL.		Area of lake surface, in square miles.	Storage capacity below lake surface, in acre-feet.
In meters.	In feet.		
*1 317.0	4 319.8	67.7	2 550 000
†1 312.5	4 305.0	55.3	.....
‡1 293.0	4 241.0	22.0	503 000

* Elevation of lip of spillway.

† Probable average elevation of lake surface during long periods of full or ultimate power production (based on study of a 62-year period).

‡ Base of the stored waters withdrawable for power production.

Its upper 78.8 ft. (the portion available for power production) has a storage capacity of 2 047 000 acre-ft. The waters stored below Elevation 1 293 m., with a maximum depth of 144.4 ft., are useful only in supporting the waters above Elevation 1 293 m. at static heads of from 141.0 to 219.8 ft. above the tail-water elevation of 1 250 m.

Because of its unusually large surface area, the evaporation losses from the lake also will be of unusual magnitude and importance. In 1911 it was judged, without special investigation, by John R. Freeman and F. P. Stearns, Members, Am. Soc. C. E., that its average annual evaporation loss would be about 55 in. depth (Mr. Freeman) and 83 in. (Mr. Stearns)—the 55 and 83 in. being the supposed gross evaporations, unreduced by the rain falling directly on the reservoir surface. The 55 in. annual evaporation loss corresponds to an average loss from the lake surface at spillway level of 275 sec-ft.; and the 83 in. corresponds to an average loss of 415 sec-ft. These two values for the average evaporation loss vary widely from each other, and even the smaller is a greater proportion of the total stream flow than is usual in water-supply investigations. Hence, as one of the steps necessary to the reliable estimation of the safe net water supply available for power production, the estimation within reasonably close limits of the true evaporation loss from the lake was of much more importance than is usual in evaporation studies.

Evaporation data in Mexico were entirely lacking; there was a very close agreement between the values of the annual evaporation depth from Lake Conchos, as estimated by the writers by five different methods; and the average yearly evaporation depth, adopted after only 2 months' local evaporation tests, was identical with the depth adopted

after 5 months' local tests; and for these reasons it is believed that this study of Lake Conchos evaporation possesses sufficient interest and value to the Engineering Profession to justify the presentation of this paper.

The paper will consist of a preliminary and general description of the data and methods used in the investigation and of the results and conclusions derived therefrom, followed (as an Appendix) by the detailed data and operations leading up to those conclusions.

The general descriptions of the data and methods used, and of the resulting conclusions, are as follows:

Data.—The evaporation data on which the investigation is based are given in full in the Appendix. The unmodified data are given in Tables 17, 34, 36 (which are Plates LXII, LXVI, and LXVII), Figs. 3 and 14, and Table 37.

Figs. 4 to 13, inclusive, Fig. 15, Plates LXIII, LXV and (Table 21) Plate LXIV deal with these data as modified and combined by the various methods of investigation used.

These data may be divided into two general groups: one includes all the obtainable data of evaporation depths and mean temperatures at stations in Texas and New Mexico; the other includes the evaporation observations initiated and carried out under the direction of the writers at La Boquilla, in conjunction with observations of mean temperature. These data comprise evaporation observations made in both land-pans and floating-pans in both groups, and include also the Piche evaporimeter observations made in Texas and New Mexico in 1887-88.

Texas and New Mexico Group.—All the data of measured evaporation depths in Texas and New Mexico were copied from such official or semi-official sources as the publications of the U. S. Weather Bureau, the U. S. Geological Survey, the University of New Mexico, *Engineering News,* Turneaure and Russell's "Public-Water-Supplies", etc., etc., the sources of all the data being given in Table 17 (Plate LXII) in connection with their presentation. Such data of evaporation depths in Texas and New Mexico, measured in pans, as were made use of in this investigation are for the following stations and durations:

At Austin, Tex.—for the 12 months of 1911.

At El Paso, Tex.—for 34 months in 1889-90 to 1892-93.

At Albuquerque, N. Mex.—for 30 months in 1900-01 and 1903-04.
At Carlsbad, N. Mex.—for 13 months in 1899 and 1901.
At Elephant Butte, N. Mex.—for 6 months in 1909-10.
At Lake Avalon, N. Mex.—for 5 months in 1909-10.

Hence the measured evaporation data of Texas and New Mexico comprise records at six places, varying from 5 to 34 months at a place, and in all aggregating 100 months. It should be noted that all the six stations, like La Boquilla and Lake Conchos, are on the Great Plateau to the east of the Continental Divide (the Western Sierra Madre).

All the evaporation records in Table 17 (Plate LXII) are given there for full years, such months as were not actually measured having been filled in by deduction. All such deduced monthly evaporation depths are indicated in Table 17 (Plate LXII), and their method of deduction and their probable errors are discussed in the Appendix.

At some of the above-named stations the evaporation measurements were made in land-pans, at others in floating-pans, and at others in both; and the pans used were of various sizes. The relative evaporation depths from pans of different sizes and from floating-pans in comparison with land-pans, are discussed fully in the Appendix; and corrections are there applied to bring the various evaporation depths to a uniform basis, so far as practicable, for all the stations, and thus permit of fair comparisons. It is believed that, as thus corrected, the various measured evaporation data in Texas and New Mexico possess sufficient accuracy to make them reliable for the object of this investigation.

The Piche evaporimeter measurements made use of in this investigation are those taken at each of eleven stations in Texas and New Mexico continuously throughout 12 months—after a somewhat disconnected study of this method had been made elsewhere for a longer period, to determine the relations between evaporation depth, temperature, vapor pressure, and barometric pressure. The evaporation depths at eleven stations, as deduced from the evaporimeter observations, also are presented in Table 17 (Plate LXII). Subsequent comparative measurements of evaporation depths in pans, at scattered stations throughout the United States, are said to indicate that the Piche method gives somewhat greater evaporation depths than are

obtained from measurements in floating-pans—though the size of pan to which that statement applies is not known.

La Boquilla Group.—The evaporation measurements made at La Boquilla (Lake Conchos), and used in this investigation, comprise the measurements of evaporation depths made at the lake in two land-pans and two (or one) floating-pans from about the middle of January to early in June, 1914. The evaporation measurements for this whole period of about 5 months were available at the close of the evaporation study; but, to prevent delay in the progress of the water-supply investigation, it was necessary to adopt tentatively a yearly evaporation depth from Lake Conchos in March, 1914, at a time when only about 2 months' evaporation measurements had been made at La Boquilla. It is worthy of remark that the average yearly evaporation depth on the lake, as adopted tentatively from the Texas and New Mexico evaporation data in conjunction with the 2 months' measured evaporations at La Boquilla, was confirmed later, and almost exactly checked, by the longer evaporation measurements extending over about 5 months.

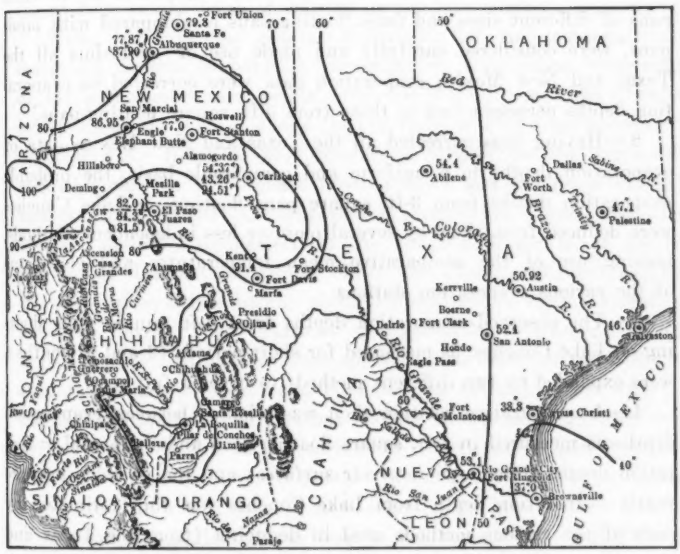
The evaporation and temperature measurements made at La Boquilla in January to June, 1914, are given in Tables 34 and 36 (Plates LXVI and LXVII).

There also were available for the investigation (see Table 37 and Fig. 14) the observed daily maximum temperatures at La Boquilla from April, 1912, to December, 1913, inclusive, the minimum temperatures from August, 1912, to August, 1913, and both maximum and minimum daily temperatures for January to June, 1914; and maximum and minimum daily temperatures at El Paso throughout 1912 and 1913.

As a rough general guide to evaporation depths in northern Mexico, there was available, also, a map showing probable lines of equal evaporation in Texas and New Mexico. This map, Fig. 3, indicates that the probable annual evaporation depth at Lake Conchos is between 70 and 80 in., presumably as measured in floating-pans.

Finally, in addition to the above-described La Boquilla, and Texas and New Mexico evaporation and temperature data, there were available the fluctuating surface elevations (and hence the net storage) of Lake Conchos, and the inflow and outflow of the lake, all as observed with reasonable closeness throughout the 7 months (practi-

cally rainless and with a nearly constant lake level) from October, 1913, to April, 1914, inclusive. These were used as data for an approximate estimate of the evaporation depth during those months, by deducting the net storage during the period from the excess of inflow


OKLAHOMA
NEW MEXICO
ARIZONA
TEXAS
CHIHUAHUA
COAHUILA
NUEVO LEON
TAMAULIPAS
SINALOA
DURANGO
Red River
Rio Grande
Pecos R.
Colorado R.
Brazos R.
Nueces R.
Fort Union
Santa Fe
Albuquerque
San Marcial
Roswell
Eagle
Fort Sumner
Elephant Butte
Hillsboro
Alamogordo
Carlsbad
Deming
Mesilla Park
El Paso
Juarez
Ascension
Casas Grandes
Ahumada
Fort Stockton
Fort Davis
Marfa
Presidio
Ojinaga
Madera
Chihuahua
Guzman
Santa Rosalia
Jimenez
Parral
Durango
Abilene
Fort Worth
Dallas
Palestine
Kerrville
Boerne
Fort Clark
Del Rio
Hondo
Eagle Pass
San Antonio
Austin
Galveston
Fort McIntosh
Corpus Christi
Laredo
Rio Grande City
Port Isabel
Brownsville
Piedras Negras
Monclova
Villa de Conchos
Mapimi
Torreon
Saltillo
Monterrey
Linares
Victoria
Tampico
Matamoros
Rio Conchos
Rio Salado
Rio Nazas
Rio Aguanaval
Rio San Juan
Rio Sabinas
Rio Soto la Marina
Rio Panuco
Rio Purificacion
Rio Mayo
Rio Yaqui
Rio Fuerte
Rio Sinaloa
Rio Mocorito
Rio Culiacan
Rio Elota
Rio Piaxtla
Rio Presidio
Rio Baluarte
Rio Acaponeta
Rio Grande de Santiago
Rio Lerma
Rio Verde
Rio Tamesi
Rio Guayalejo
Rio Tamesi
Rio Tuxpan
Rio Cazones
Rio Tecolutla
Rio Nautla
Rio Actopan
Rio Jamapa
Rio Papaloapan
Rio Coatzacoalcos
Rio Tonala
Rio Grijalva
Rio Usumacinta
Rio Champoton
Rio Hondo
Rio Candelaria
Rio Mamantel
Rio Palizada
Rio Chumpan
Rio Tulijapa
Rio Tacotalpa
Rio Teapa
Rio Puxcatan
Rio Tonala
Rio Zanapa
Rio Tehuantepec
Rio Coatan
Rio Suchiate
Rio Motagua
Rio Ulua
Rio Chamelecon
Rio Aguan
Rio Patuca
Rio Segovia
Rio Coco
Rio Grande de Matagalpa
Rio Escondido
Rio San Juan
Rio Colorado
Rio Sarapiqui
Rio Frio
Rio Sixaola
Rio Tarire
Rio Reventazon
Rio Pacuare
Rio Matina
Rio Estrella
Rio Banano
Rio Chirripo Grande
Rio Sierpe
Rio Terraba
Rio Grande de Terraba
Rio Coto
Rio Grande de Coto
Rio Grande de Chiriqui
Rio Chiriqui Viejo
Rio Changuinola
Rio Tigre
Rio Calovebora
Rio Chagres
Rio Trinidad
Rio Cascajal
Rio Cocle del Norte
Rio Cocle del Sur
Rio Grande de Penonome
Rio Grande de Cocle
Rio Grande de Los Santos
Rio Tabasara
Rio Chucunaque
Rio Tuira
Rio Chepo
Rio Bayano
Rio Mandinga

Methods.—The general method used in the Appendix to estimate a safe yearly evaporation depth from Lake Conchos consists of the following principal steps:

1.—The available data of the comparative evaporation depths from pans of different sizes and from floating-pans as compared with land-pans, were considered carefully and made use of—and thus all the Texas and New Mexico evaporation data were corrected to evaporation depths corresponding to those from 3-ft. square floating-pans.

2.—Having thus corrected all the Texas and New Mexico data of evaporation depths to a uniform and comparable basis, the probable evaporation depths from 3-ft. square pans floating on Lake Conchos were deduced from them by several more or less independent methods, making use of the comparative mean temperatures and elevations of the various evaporation stations.

3.—The observed evaporation depths from 3-ft. square pans floating on Lake Conchos, as measured for short periods (2 and 5 months), were expanded by two different methods to cover a full year.

4.—A comparative relation then was adopted between evaporation depths as measured in 3-ft. square floating-pans and the actual evaporation depths from large reservoir surfaces; and the probable average yearly evaporation depth from Lake Conchos was thus estimated by each of the various methods used in deducing (from the Texas and New Mexico and the La Boquilla data) the yearly evaporation depth from 3-ft. square pans floating on the lake.

5.—The actual evaporation depth from the surface of Lake Conchos was estimated (as closely as practicable) by using the excess of inflow over outflow plus waters stored for a 7-month period—during which continuous observations had been made of the fluctuating elevations of the lake surface, and continuous measurements of the inflow at the head of the lake, at Pilar de Conchos; during which period there had been practically no inflows (because of almost no rain) between Pilar de Conchos and the La Boquilla Dam; and during which all outflow through the dam (and all losses of stored waters except evaporation losses) had been measured as carefully as practicable.

6.—Finally, from a consideration of the evaporation depth as estimated by the last-named method (which is a full-sized check method) and the evaporation depths as estimated by the other methods, the

adoption was made of a safe average yearly evaporation depth from the surface of Lake Conchos.

This adopted evaporation depth evidently is a gross depth; and, for use in the water-supply investigation, requires to be reduced by the average yearly depth of rainfall on the lake surface.

With the foregoing general explanation of the principal steps comprised in this evaporation investigation, estimations were made of the probable average yearly evaporation depth on Lake Conchos by the following three methods (two of them with two variations):

Method A.—Relations were determined between the monthly evaporation depths (in 3-ft. square floating-pans) and the mean monthly temperatures, at the six evaporation stations in Texas and New Mexico; and (by comparison of the evaporation depths at the six stations of different elevations, all in the Plateau Region) between the evaporation depths at any given temperature, at various elevations above sea level. These relations of evaporation to temperature and evaporation to altitude were then applied to the observed and corrected evaporation data of Texas and New Mexico, in conjunction with the known elevation of Lake Conchos and its known mean monthly temperatures (which had been observed for a little more than 2 years); and thus the probable mean yearly evaporation depth was estimated, as measured in a 3-ft. square pan floating on the lake.

Method B.—Method *B* was used in two variations, *a* and *b*:

(*a*) From the available evaporation data of Texas and New Mexico, the proportion of a full year's evaporation depth which occurs during the 6 months, January-June, was determined; and from this average proportion and the observed evaporation depths at La Boquilla for the (nearly) 6 months, January-June, 1914, the probable average evaporation depth from the lake for a full year was estimated.

(*b*) From the temperature-evaporation relations of Method *A*, the Texas and New Mexico observed evaporation depths and temperatures, and the known mean monthly temperatures at Lake Conchos during the 6 months, January-June, the corresponding theoretical evaporation depths at the lake during the 6 months were estimated. These theoretical evaporation depths were compared with the evaporation depths actually measured at the lake during the same period, and the percentage relation between theoretical and actual evaporation depths thus obtained was applied to the average annual evaporation

depth previously estimated for the lake by the temperature-evaporation study of Method *A*. The average yearly evaporation depth resulting from Method *A* was thus modified and made more reliable by the use of the nearly 6 months' local evaporation measurements made at the lake.

(A third variation of Method *B* was attempted, using short-period evaporation and temperature observations at El Paso, Tex., made simultaneously with those at Lake Conchos; but this third variation proved to be worthless, because of the shortness of the El Paso record and the apparent vagaries of the temperature there at that time.)

Method C.—Method *C* made use of the estimated excess of inflows into Lake Conchos over outflows plus waters stored, expressing that excess (or evaporation loss) in terms of depth on the lake surface, and then expanding this estimated gross evaporation depth from the lake surface for the 7-month period into a full yearly period and depth, by the sub-methods, *a* and *b*, described under Method *B*.

This concludes the general description of the methods used.

Results and Conclusions.—The results and conclusions of this evaporation investigation may be divided into two groups, subsidiary and principal, with subjects as follows:

Subjects of Subsidiary Results and Conclusions.

(*a*) Relative evaporation depths from evaporation pans of different sizes;
(*b*) From floating-pans in comparison with land-pans;
(*c*) From large reservoirs in comparison with floating-pans;
(*d*) Relations between evaporation depth and mean temperature;
(*e*) Relations between evaporation depth and elevation above sea level.

The principal result and conclusion is the main subject of inquiry of the investigation—the probable average annual evaporation depth from the surface of Lake Conchos, as estimated by the three (or rather five) more or less independent methods already described.

Subsidiary Conclusions.

The subsidiary conclusions of the investigation are given in Table 2.

In Table 2, the larger evaporation losses from smaller than from larger pans (shown by (*a*)) and from floating-pans than from large

TABLE 2.—SUBSIDIARY CONCLUSIONS.

(*a*)—Evaporation depth from a 2-ft. square pan	= about	108%	of that from a 3-ft. square pan.						
Evaporation depth from a 2½-ft. square pan	= "	104%	"	"	"	"	3	"	"
Evaporation depth from a 3-ft. square pan	= "	100%	"	"	"	"	3	"	"
Evaporation depth from a 4-ft. square pan	= "	93%	"	"	"	"	3	"	"
Evaporation depth from a 5-ft. square pan	= "	86%	"	"	"	"	3	"	"
Evaporation depth from a 6-ft. square pan	= "	80%	"	"	"	"	3	"	"

(*b*)—Evaporation depth from a floating pan = about 80% of that from a land-pan of the same size.

(*c*)—Evaporation depth from a large reservoir = about 62% of that from a 3-ft. square pan floating thereon.

(*d*)—In the Great Plateau Region, and at elevations above sea level of 3 000 to 5 000 ft and monthly evaporation losses (from 3-ft. square floating-pans) of 2 to 10 in. the increases in the evaporation depth are nearly in direct proportion to the increases in mean monthly temperature; but at lower elevations (3 000 to 600 ft.) the evaporation depth increases with the mean temperature, but much less rapidly than in direct proportion.

(*e*)—In the Great Plateau Region, and at elevations above sea level of 600 to 5 000 ft. and monthly mean temperatures higher than 70° Fahr., the increases in the evaporation depth at any given mean temperature are directly proportional to increases in elevation; but at monthly mean temperatures lower than 70° Fahr., and at the lower elevations, the evaporation depth at any given mean temperature increases with the increase in elevation, but less rapidly than in direct proportion.

reservoirs (shown by (*c*)), presumably are due to the greater heating up of the water in smaller than in larger pans, and in pans than in reservoirs, because of the greater confinement and less circulation of the smaller masses of water, and to the deterrent effect on the evaporation losses of the better and more complete "vapor blankets" existing over larger pans and over reservoirs.

The smaller evaporation losses from floating-pans than from land-pans, shown by (*b*), presumably are due to the greater deterrent effect of the better "vapor blanket" over floating-pans; and to the lower temperature of the water in floating-pans, caused by the influence of the still cooler water in the reservoir.

Subsidiary Conclusions (*d*) and (*e*) are formed by inspection from Tables 23, 24, and 25 of the Appendix; and from Fig. 4—which shows by diagrams the relations between evaporation depths and mean temperatures and elevations.

The subsidiary conclusions presented in Table 2 are worked out and discussed in full in the Appendix, where they are shown more completely by tables and diagrams.

Principal Conclusions.

The conclusions as to the average yearly evaporation depth from Lake Conchos, as estimated by the three more or less independent methods and their two variations, are shown in Tables 3 and 4.

The evaporation depths of Tables 3 and 4 are derived and discussed in full in the Appendix.

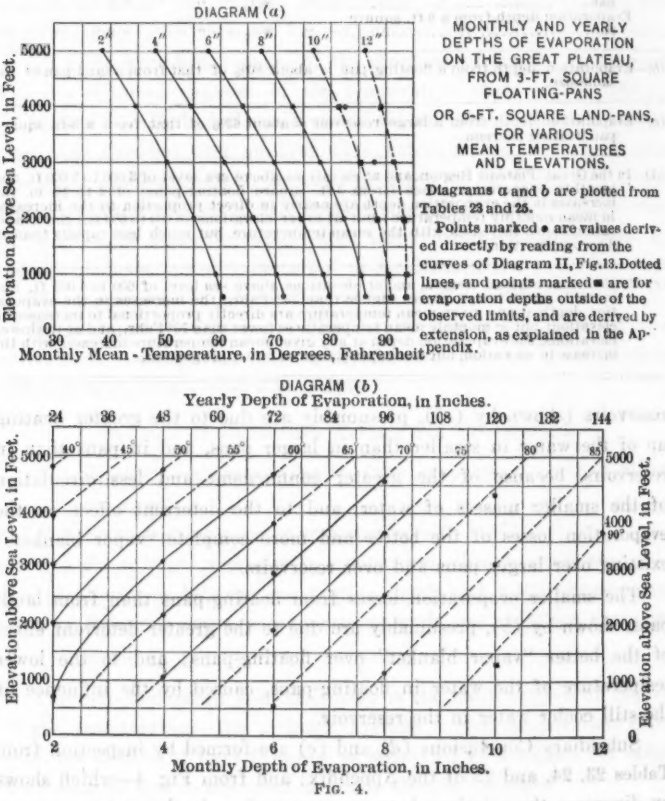


Fig. 4.

It is apparent from the percentage relations of Table 4 that the evaporation depth from the surface of Lake Conchos, estimated by Method *A*, is in very close agreement (within 2.4%) with the depth estimated by Method *B*; but that the depth estimated by Method *C* is nearly 12% greater than that by Method *B*.

TABLE 3.—FIVE ESTIMATED VALUES OF THE AVERAGE YEARLY EVAPORATION DEPTH FROM THE SURFACE OF LAKE CONCHOS.

By Method A:	Estimating the evaporation from measured yearly pan evaporations in Texas and New Mexico, modified for differences in mean temperature and in elevation.....	52.5 in.
By Method B_a:	Estimating the evaporation from the 6 months' (nearly) measurements of pan evaporation at Lake Conchos; expanded to a full year by the percentage relation adopted (from the Texas and New Mexico evaporation data) between the evaporation depth for those 6 months and that for a full year.....	54.1 in.
By Method B_b:	Same as B_a, except that the observed depth of pan evaporation at La Boquilla for 6 months is expanded to a full year's depth by evaporation-temperature relations.....	53.4 in.
By Method C_a:	Estimating the evaporation from the excess of the inflows over the outflows plus the increase in storage of the lake for 7 months; expanded into a full year's evaporation depth by the adopted percentage relation borne by those 7 months' evaporation depth to that of a full year (as in Method B_a).....	57.7 in.
By Method C_b:	Same as Method C_a, except that the evaporation depth for the 7-month period is expanded to that for a full year by the use of evaporation-temperature relations (as in Method B_b).....	59.3 in.

TABLE 4.—SUMMARY AND COMPARISON OF THE FIVE VALUES OF YEARLY EVAPORATION FROM LAKE CONCHOS.

Method of estimating.	Average yearly gross evaporation depth from the lake.
A.....	52.5 in. = 100.0%
B_a.....	54.1 " = 103.0%
B_b.....	53.4 " = 101.7%
Mean of B_a and B_b.....	53.75 " = 102.4%
Mean of A and Mean B.....	53.13 " = 101.2%
C_a.....	57.7 " = 109.9%
C_b.....	59.3 " = 113.0%
Mean of C_a and C_b.....	58.50 " = 111.5%
Mean of A, Mean B, and Mean C.....	54.92 " = 104.6%
Mean of A, B_a, B_b, C_a and C_b.....	55.40 " = 105.5%
Adopted value.....	55 0 in. = 104.8%

The evaporation depth resulting from Method B is more reliable and accurate than that from Method A, because the former includes the consideration of, not only all the data used in Method A, but also the local pan observations of evaporation made at Lake Conchos. For reasons about to be given, the evaporation depths resulting from Method C are known to be somewhat greater than the true depths, and indeed are regarded rather as reasonably close (and full-sized

model) rough checks on the results of Method *B*, than as equally careful determinations of the evaporation depth by another method.

Hence it is believed that the most reliable and accurate value of the average yearly evaporation from the surface of the lake is the depth of 53.75 in. (1.365 m.) resulting from Method *B*.

The principal reasons for believing the results of Method *C* to be greater than the true value of the evaporation depth are as follows:

(1)—During the 3 months from the middle of June to the middle of September, 1913, the surface of Lake Conchos was raised 100 ft. by the flood waters impounded, from a maximum depth of 29.5 ft. to one of 129.5 ft.; and its surface area was increased about 15½ sq. miles, from less than 0.4 to about 15.8 sq. miles. The area and elevation of the lake then remained nearly constant until the beginning of the next flood season, in June, 1914. Method *C* is based on a consideration of 7 months of this period, from October, 1913, to April, 1914, inclusive.

Even though the bottom of the reservoir or lake is to a large extent rocky and apparently but slightly absorptive of the water (see Fig. 2), during the 7 months, October to April, there must nevertheless have been some water lost by absorption in the 15½ sq. miles of reservoir bottom inundated recently and for the first time. This obviously is true of the soil-covered portions, and must be true of the bare rock portions also, because the seepage springs fed by the lake (most of which springs made their first appearance during these 7 months, October to April) are known to reach, in their course from lake to springs, depths of at least 2 000 ft. below the lake surface. In Method *C*, however, no attempt was made to estimate separately such absorbed water, which in consequence was treated as if part of the water evaporated. Hence, for this reason (and for some others also), the evaporation depths resulting from Method *C* are known to be greater than the actual ones.

It is apparent from Table 4 that the yearly evaporation depth by Method *C* exceeds that by Method *B* by only (58.50 — 53.75) = 4.75 in. This excess, with an average void content of the material forming the reservoir bottom of, say, 20%, would require an average depth below the reservoir bottom of only 24 in. for its entire absorption. Again, during the 7 months, October to April, the lake remained nearly constant at a level of about 1 289 m., an area of about 16.2

sq. miles, and about 346 000 acre-ft. of impounded water. The excess depth of 4.75 in. over the 16.2 sq. miles equals about 4 100 acre-ft., or only 1.2% of the impounded water. It has been said by Fteley and Herschel (referring presumably to some of the reservoirs of the New Croton Aqueduct system) that some reservoirs on being emptied return from ground storage as much as 10% of their capacity.*

It is evident from either view of the question that there is no improbability in the assumption that 4.75 in. depth of water, assumed by Method *C* as part of the water evaporated, instead was absorbed by the bottom of the reservoir during the 7 months considered.

(2)—During the 7-month period used in Method *C*, efforts were made to measure all seepage losses from the lake. Certain losses were unmeasurable, however, such as ground evaporations from wet soil between the springs and the measuring weirs, and such seepages as may enter the river below its water surface. In an attempt to cover such unmeasurable seepages, 10% was added arbitrarily to the sum of the measured seepages; but it is not unlikely that the 10% so assumed may be too little.

(3)—At lake level 1 289 m., the area of the lake was about 16.2 sq. miles and its shore line about 86.4 miles, or its equivalent mean diameter or width about $[(16.2 \div 86.4) \times 4] = 0.75$ mile. At the average future lake level of 1 312.5 m., the area will be about 55.3 sq. miles, the shore line about 152 miles, and the equivalent mean width about 1.45 miles, or nearly twice as great. It may be that the greater width in the future will give a better "vapor blanket", and one more deterrent to evaporation losses—and hence that the yearly evaporation depth throughout the future may be less than that estimated from the 7 months, October to April, used in Method *C*. Such smaller evaporation depth because of the larger lake surface is regarded as very doubtful, however.

(4)—In addition to the evaporation from the lake surface, there always will be some evaporation from the narrow strip along the shore which soaks up water by capillary action. During the 7 months considered in Method *C*, however, with the lake level nearly constant at about 1 289 m., such evaporation losses were relatively much greater

* However, part of the 10% presumably was from the raised ground-water outside the horizontal limits of the water stored in the reservoir.

than they will be throughout the future, because at 1 289 m. there are about 5.3 miles of shore line per square mile of water surface, and at the average lake level of 1 312.5 m. there will be only about 2.7 miles per square mile, or only 51% as much.

For the several reasons given, it is believed that the somewhat greater evaporation depths resulting from Method *C* may be disregarded, at least partly; and that a mean of the five values of evaporation depth obtained by the various methods may be adopted with safety as being not too low. Hence the mean of the five values, say, 55 in., or 1.40 m., was adopted as a safe mean annual evaporation depth from the surface of Lake Conchos throughout the future. This 55 in. is a gross annual evaporation depth, and in use will be reduced by the depth of rain falling directly on the reservoir surface, which amounts to an average of about 12.0 in. per year.

It should be added that the foregoing conclusion is based on all the La Boquilla or Lake Conchos evaporation data secured up to the beginning of the heavy floods of June, 1914; but that the 55 in. thus estimated from the use of all the data secured is in practically exact agreement with the results estimated from the shorter period of 2 months, beginning in January and ending in March, 1914. This close agreement is shown by the comparison in Table 5.

The evaporation observations at the lake were terminated early in June, 1914, by heavy storms and floods, which swamped the floating-pans, made the inflows to the lake unmeasurable, etc.

From an evaporation-temperature study (given in the Appendix), it was ascertained that the monthly percentage distribution of the year's gross evaporation depth is about the same at the lake as at the evaporation stations in Texas and New Mexico used in this investigation; and the probable percentages of the yearly evaporation occurring each month thus were adopted. These adopted monthly percentages and their corresponding monthly evaporation depths in inches are given in Table 6. Also (as of interest in showing the characteristics of the local climate), there are included in that table the average depth of rain falling on the reservoir surface each month and the corresponding net evaporation depth.

It is worthy of repetition here that in 1911 Mr. John R. Freeman adopted 55 in. as the probable average yearly gross evaporation

TABLE 5.—COMPARISON OF VALUES FOR PERIODS OF TWO AND FIVE MONTHS.

Method.	Yearly Gross Evaporation Depth from Lake Conchos.			
	From 2 months' pan observations at Lake Conchos.		From 5 months' pan observations at Lake Conchos.	
	Inches.	Percentage.	Inches.	Percentage.
A	52.5*	= 100.0	52.5*	= 100
B_a	52.9†	= 97.9	54.1†	= 100
B_b	52.4†	= 98.2	53.4†	= 100
C_a	55.4‡	= 96.2	57.7‡	= 100
C_b	56.2‡	= 94.8	59.3‡	= 100
Mean	53.88	= 97.3	55.40	= 100
Adopted	55.0	= 100.0	55.0	= 100

* Based on no pan evaporations at the lake, but on more than a year's mean monthly temperatures there.

† Based on Method A, as modified by 2 months' and 5 months' pan evaporations at the lake.

‡ Based on 4 months' and 7 months' lake observations, expanded into a full year by Methods B_a and B_b, respectively.

TABLE 6.—ADOPTED AVERAGE MONTHLY GROSS AND NET EVAPORATION DEPTHS FROM LAKE CONCHOS.

Month.	Gross Evaporation Depths.		Rain Falling Directly on Lake Surface.		Net Evaporation Depths from Lake Surface.	
	Percentage.	Inches.	Percentage.	Inches.	Inches.	Percentage.
(1)	(2)	(3)	(4)	(5)	(6) (3—5)	(7)
Jan.	4.8	= 2.6	1.0	= 0.1	2.5	= 5.8
Feb.	5.6	= 3.1	2.5	= 0.3	2.8	= 6.5
Mar.	7.4	= 4.1	2.5	= 0.3	3.8	= 8.8
Apr.	9.4	= 5.2	3.0	= 0.4	4.8	= 11.1
May	11.5	= 6.3	4.0	= 0.5	5.8	= 13.5
June	11.7	= 6.4	13.7	= 1.7	4.7	= 10.9
July	12.2	= 6.7	17.7	= 2.1	4.6	= 10.7
Aug.	11.1	= 6.1	21.5	= 2.6	3.5	= 8.2
Sept.	9.4	= 5.2	20.2	= 2.4	2.8	= 6.5
Oct.	7.7	= 4.2	7.7	= 0.9	3.3	= 7.7
Nov.	5.1	= 2.8	3.4	= 0.4	2.4	= 5.6
Dec.	4.1	= 2.3	2.8	= 0.3	2.0	= 4.7
Year	100.0	= 55.0 or 1.40 m.	100.0	= 12.0 or = 0.305 m.	43.0 or 1.09 m.	= 100.0

depth from Lake Conchos, and this without special study or investigation, or local observations, but merely by the exercise of judgment.

This contribution to the existing engineering information of the evaporation losses from reservoirs is offered with a full realization that many other meteorological conditions than mean temperatures and elevations have more or less influence on the depth of evaporation; and that whenever time and opportunity permit of their accumulation, data of humidities, wind velocities and frequencies, etc., also are desirable as aids in estimating probable future evaporation losses.

However, it is believed to be apparent as the result of this evaporation study that whenever time or opportunity is lacking for the collection of full local meteorological data (as so frequently is the case in engineering investigations), then reliable conclusions as to monthly and yearly evaporation depths from reservoirs may be drawn without other data than observed pan evaporations elsewhere (at places having similar characteristics of climate) modified skilfully by the comparative mean temperatures and elevations above sea level.

Also, it is believed that for places within the limits of the Great Plateau, from Mexico on the south to perhaps Colorado and Utah on the north, fair general approximations to local monthly and yearly evaporation depths may be read directly from Fig. 4 without other data than local mean temperatures and elevations.

The data on which this study is based, and the working out of this investigation, are given in full in the Appendix.

APPENDIX

Evaporation Pans at Lake Conchos.—The locations and exposures of the pans are explained sufficiently in Table 34 (Plate LXVI). The depths of water in the pans were read frequently (part of the time daily), by an ordinary rule, to $\frac{1}{32}$ in. As far as practicable, the pans were kept filled with water to a depth of about 10 in., or about 2 in. below their rims. The depths were corrected for the infrequent rainfalls by near-by rain gauges.

Subsidiary Conclusion (*a*):

Relative Evaporation Depths from Pans of Different Sizes.

It has been noted that when evaporation measurements are made in pans, the depths evaporated are greater, under the same climatic conditions, if the measurements are made in small than in large pans.

Experiments to determine the relative evaporation depths from pans of different sizes have been made by the U. S. Weather Bureau.*

Professor Bigelow states that the evaporation depths from pans are given by the formula:

$$E_o = C_2 \times \frac{e_s}{e_d} \times \frac{d\,e}{d\,S} \times (1 + 0.070\,w)$$

in which

E_o = the evaporation-depth in 4 hours;
e_s = the vapor pressure at the water temperature;
e_d = the vapor pressure at the dew-point temperature;
$\frac{d\,e}{d\,S}$ = the coefficient from the physical table;
w = the wind velocity, in kilometers per hour;
and C_2 = a variable, depending on the size of the evaporation pan.

He states, further, that, based on experiments made by the U. S. Weather Bureau at the Salton Sea and elsewhere, various values of C_2 are as follows:

For 2-ft. square pans	$C_2 = 0.042$
" 4- " " "	" = 0.036
" 6- " " "	" = 0.031
" large lake surfaces	" = 0.024

From these values of C_2, the curve of Fig. 5 is plotted; and from that curve Table 7 is made up.

* The conclusions therefrom, as summarized by Frank H. Bigelow, Professor of Meteorology, U. S. Weather Bureau, in charge of the Climatological Division, are published in *Engineering News*, Vol. 63, pp. 694–5.

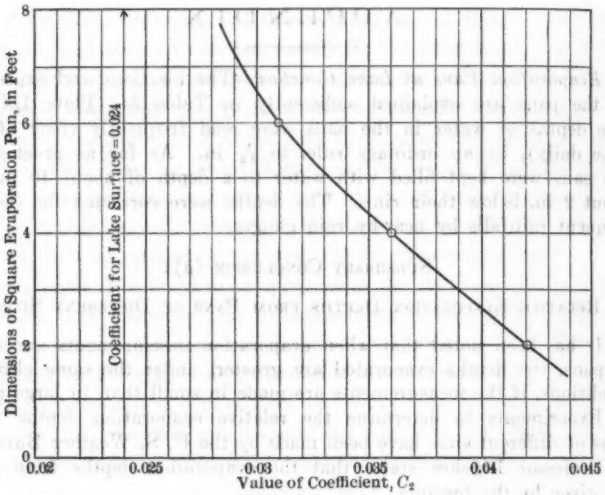


FIG. 5.

TABLE 7.—RELATIVE EVAPORATION DEPTHS FROM PANS OF VARIOUS SIZES.

Size of pan.	Coefficient, C_2.	Relative evaporation depth.
2 ft. square.....	0.0420	108% of the depth from a pan 3 ft. square.
2½ " "	0.0405*	†104% " " " " " 3 " "
3 " "	0.0389*	100% " " " " " 3 " "
4 " "	0.0360	93% " " " " " 3 " "
5 " "	0.0333*	86% " " " " " 3 " "
6 " "	0.0310	80% " " " " " 3 " "

* Interpolated from the curve of Fig. 5.

† By a clerical error, this was used as 105% in the succeeding parts of the investigation—in which this error has been left uncorrected because unimportant.

In a full consideration of the effect of the size of the pan on the depth of evaporation, it is evident that, not only the horizontal dimensions of the pans should be considered, but also the depths. However, available records of evaporation tests, though usually stating the horizontal dimensions of the pans, more often than not fail to mention their depths; and the depths of most of the pans used in the Texas and New Mexico evaporation tests which form the basis of this study are unknown. Hence a consideration in this study of the effect of depth of pan on evaporation depth necessarily had to be omitted.

The standard evaporation pan of the U. S. Geological Survey is believed to be 3 ft. square and 18 in. deep; and during evaporation tests it is supposed to be kept filled with water to within about 3 or

4 in. of the rim. The Texas and New Mexico pans are believed to have been generally less than 18 in. deep. The pans used for the Lake Conchos evaporation tests were 3 ft. square and only 12 in. deep; and they were kept filled with water to approximately 2 in. below the rim. It now seems that it might have been preferable if they had been 18 in. deep, the standard depth of pan of the U. S. Geological Survey.

However, the purposes of the Rio Conchos investigation were to estimate an evaporation loss which would be certainly not below the actual loss, and (by deducting the evaporation loss from the gross stream flow) to estimate a net stream flow available for power production which would be certainly not greater than the actual; and for these purposes pans 12 in. deep probably were safer and more conservative than deeper ones would have been, as it is believed that shallower pans give greater evaporation losses than deeper ones. Also, the 12-in. depth is believed to be more nearly in agreement with that of the Texas and New Mexico pans, from which the evaporation depth at Lake Conchos was being deduced by modifications for differences in mean temperatures and in elevations.

Subsidiary Conclusion (*b*):

Relative Evaporation Depths from Land-Pans and from Near-by Floating-Pans.

It has been found by observation that the depth of evaporation from floating-pans is about 80% of that from near-by land-pans of the same size. A few instances showing this relation are as follows:

*Granite Reef, Ariz.**—(Presumably at least a year's observations.)

A year's evaporation from 4-ft. square, land-pan = 115.18 in. = 100%
" " " " 4-ft. " floating-pan = 97.74 in. = 84.8%

*California, Ohio.**—(Presumably at least a year's observations.)

A year's evaporation from 3-ft. square, land-pan = 61.83 in. = 100%
" " " " 4-ft. " floating-pan = 45.99 in. = ——
" " " " 3-ft. " floating-pan (equals 100% ÷ 93% = 107.5% of that from a 4-ft. square, floating-pan) = 49.4 in. = 80.0%

Coyote, Cal. (*Near San José*).—These observations were made under the writers' supervision, and consist of the evaporations from two pans floating on the Laguna Seca (a shallow natural lake), as compared with the evaporations from three near-by land-pans. These observations covered 2 years.† The briefest summary of the Laguna Seca evaporation observations is as follows:

* *Engineering News*, Vol. 63, p. 695.

† Abstracts of these observations (with additional evaporation data) are published in *Engineering News*, Vol. 67, pp. 380–83.

	1904.	1905.
A year's evaporation depth from a 3-ft. square, land-pan (mean of three pans).....	56.6 in. = 100%	56.9 in. = 100%
A year's evaporation depth from a 3-ft. square, floating-pan (mean of two pans).....	41.2 in. = 72.8%	48.3 in. = 85.0%

Mean of the 2 years: floating-pan = 78.9% of land-pan.

*Salton Sea, Cal.**—(Presumably at least a year.)

A year's evaporation depth from a 2-ft. square, land-pan.....	= 164.50 in. = ——
A year's evaporation depth from a 4-ft. square, land-pan (equals 93 ÷ 108 = 86.1% of that from a 2-ft. square land-pan.)	= 141.6 in. = 100%
A year's evaporation depth from a 4-ft. square, floating-pan (mean of two pans, in tower 2 ft. above surface of sea)....	= 107.55 in. = 76.0%

TABLE 8.—SUMMARY OF EVAPORATION TESTS.

Evaporation station.	Evaporation depth from a floating-pan, as a percentage of that from a near-by land-pan of the same size.
Granite Reef, Ariz.	84.8%
California, Ohio	80.0%
Coyote, Cal.	78.9%
Salton Sea, Cal.	76.0%
Mean of the four	79.9%

Based on the measured instances in Table 8 (and especially on the careful Coyote tests made under the writers' own supervision, and comprising the records of two floating-pans for 2 years) the depth of evaporation from a floating-pan was adopted as 80% of that from a near-by land-pan of the same size. The 80% relation appeared to be confirmed, also, by values then worked up from the Lake Conchos tests, as shown in Table 9.

TABLE 9.—PARTIAL LAKE CONCHOS TESTS.

Periods: (1 and 2 months).	EVAPORATION FROM:	
	Land-pan (Pan No. 1).	Floating-pan (Pan No. 2).
After 1 month's evaporation (to middle of February, 1914)...	100%	79.7%
" 2 months' " (" " " March, ")...	100%	80.1%

* *Engineering News*, Vol. 63, p. 695.

However, the remaining portion of the Lake Conchos tests (which was not examined as to the 80% relation until the present time, in connection with the writing of this paper) shows a disagreement with the 80% relation, as may be seen in Tables 10 and 11.

TABLE 10.—PARTIAL LAKE CONCHOS TESTS.

Periods: (1 and 2 months).	EVAPORATION FROM	
	Land-pan (Pan No. 3).	Floating pan (Pan No. 4).
After 1 month's evaporation (at middle of February, 1914)...	100%	85.5%
" 2 months' " (" " " March, ")...	100%	87.0%

(Pan No. 4 then was discontinued because it was very frequently swamped by waves.)

TABLE 11.—PARTIAL LAKE CONCHOS TESTS.

Periods: (3, 4, and 5 months).	EVAPORATION FROM:	
	Land-pan (Pan No. 1).	Floating-pan (Pan No. 2).
After 3 months' evaporation (at middle of April, 1914).....	100%	84.8%
" 4 " " (" " " May, ").....	100%	87.8%
" 5 " " (" " " June, ").....	100%	86.5%

As mentioned already, the disagreements at La Boquilla with the 80% relation were not realized until the present time. It is believed, however, because of the much longer tests made for this relation at Coyote and elsewhere, that the adoption of the 80% relation still is justified.

SUBSIDIARY CONCLUSION (*c*):

RELATIVE EVAPORATION-DEPTH FROM LARGE RESERVOIRS, AS COMPARED WITH THAT FROM 3-FT. SQUARE PANS FLOATING THEREON.

For some time past it has been recognized more or less generally that the evaporation losses from large reservoirs usually have been over-estimated, and that such losses are materially less than the evaporation depths measured in pans, even when the pans are floating on the reservoir.

In 1907 Professor Bigelow discussed this subject as follows:

"Large bodies of water are differently affected by the wind from small bodies, such as can be experimented with in the shape of tanks and pans. As the air moves across a reservoir and gradually becomes charged with moisture, its rate of absorption diminishes, and the average rate of evaporation for a broad surface is therefore less than

for small surfaces. For this reason, the formula for evaporation cannot be put on a sound basis, without taking account of large water surfaces as well as of small. The conditions afforded by the Salton Sea are peculiarly suitable for the investigation of the laws of evaporation."

Professor Bigelow reached the following conclusions*:

"* * * in the arid regions of the West it seems probable that a lake or reservoir evaporates about five-eights as fast as an isolated pan placed outside of the vapor blanket; in other words, this vapor blanket seems to conserve about three-eights of the water that would otherwise be lost by the evaporation. It is important that similar experiments with towers be made in the central and eastern portions of the United States, in the prevailing damp climates, to discover whether similar rules can be applied in practise. A careful campaign on the theory of evaporation is evidently demanded to elucidate this complex function of the evaporation of water in the open air, and it is probable that several years will be required in order to bring it to a satisfactory conclusion."

Again, in 1910, in an article† by Professor Bigelow, his conclusions in this matter are given. At the close of that article he states as follows:

"These data [the Salton Sea evaporation tests] indicate a nearly uniform rate of evaporation over the area of the water, beginning a short distance from the shore. If 70 ins. is admitted as the amount evaporated from the Salton Sea,‡ there remains 38 [38.65?] ins. as the difference between the [evaporation from the] water in the sea and that in the lower swinging pan§. It was not possible to float pans in the waves of the Salton Sea, and it is not easy to take the step in the research just indicated in order to arrive at the true coefficient [of evaporation] for the water of the Salton Sea. Our computations on the temperatures are not yet completed regarding this point, and we are conducting another experiment|| with a series of pans 2 ft., 4 ft., 6 ft. and 12 ft. in diameter. By computing with the formula,¶ assuming values of $C_v = 0.021, 0.023, 0.025, 0.027, 0.029$, and using the observed temperatures of the Salton Sea surface water, it is now probable that the coefficient for a large water surface is $C_v = 0.024$."

Professor Bigelow states elsewhere in the same article:

"The automatic record indicates that the Salton Sea has fallen about 4.60 ft. annually. * * * It is probable that the inflow from the New and Alamo rivers, together with the precipitation flowing in from the surrounding country, amounts to something like 1.5 ft. [annually]. [Hence,] allowing for the inflow, we have a little more or less than 6.0 ft. by evaporation [annually from the Salton Sea]".

* *Monthly Weather Review*, 1908.

† *Engineering News*, Vol. 63, pp. 694–695; this being the article which already has been referred to in discussing the relative depths of evaporation from pans of different sizes, and from land-pans as compared with floating-pans.

‡ See additional quotation following this one.

§ Two feet above the surface of the sea.

|| Which it is believed was discontinued before completion.

¶ Given on p. 1851.

From Fig. 5 and Table 7, the value of C_2 for a 3-ft. square pan* is about 0.0389. This value, compared with the value of 0.024, gives the depth of evaporation annually from a large water surface as $(0.024 \div 0.0389) = 61.7\%$ of that from a 3-ft. square pan floating thereon.

In an extensive table in the foregoing article Professor Bigelow gives many data of the Salton Sea evaporation tests, among them those in Table 12.

TABLE 12.—SALTON SEA TESTS.

Size and location of pan.	TOWER NO. 2, 500 FT. AT SEA.		TOWER NO 4, 7500 FT. AT SEA.	
	Inches.	Percentage.	Inches.	Percentage.
Annual evaporation depth from 4-ft. square* pan. 2 ft. above surface of sea.....	108.65	100.0	106.45	100.
Annual evaporation depth from 4-ft. square* pan, 45 ft. above surface of sea.....	137.71	126.7	140.02	131.6

* Pan mentioned as 4 ft. "in diameter", but believed to have been 4 ft. "square", instead.

From Table 7, the evaporation depth from a 3-ft. square pan is $(0.0389 \div 0.0360) = 108.0\%$ of that from a pan 4 ft. square, hence, if the lower pans of Table 12 had been only 3 ft. square, their evaporation depths should have been:

Lower pan in Tower No. 2 $= (108.65 \times 1.08) = 117.5$ in.
" " " " No. 4 $= (106.45 \times 1.08) = 115.0$ "

From the 70 in. believed by Professor Bigelow to have been the annual evaporation depth from the surface of the Salton Sea, and the assumption that the evaporation depths from pans 2 ft. above the surface of the sea were the same as if the pans had been floating in the sea instead, the following relations result:

Evaporation depth from the surface of Salton Sea, as a percentage of that from a pan 3 ft. square floating thereon .....

From Tower No. 2 $= (70 \div 117.5) = 59.6\%$
From Tower No. 4 $= (70 \div 115.0) = 60.9\%$
Mean = ... 60.3%

This value varies but little from the 61.7% derived from the $C_2 = 0.024$ adopted tentatively by Professor Bigelow, presumably from a fuller and more general consideration, based on more complete data.

* Presumably, from the context, a floating-pan.

The following quotations show that there has been some general recognition in the recent past of the smaller evaporation depths from reservoirs than from pans, and from large than from small pans. All the opinions quoted seem to be based on the Salton Sea evaporation tests.

Mansfield Merriman, M. Am. Soc. C. E., states:*

"The evaporation from a pan† 2 feet in diameter‡ is about 75%, that from a pan 4 feet in diameter is about 50%, and that from a pan 6 feet in diameter is about 30% greater than the evaporation from a large pond or lake."

By the comparison in Table 13, the foregoing relations are seen to be identical with those of Table 7.

TABLE 13.—COMPARATIVE EVAPORATION DEPTHS FROM PANS OF VARIOUS SIZES.

Size of pan.	Comparative percentages.
2 ft. square	175% = 108%
3 " "	*162 = 100
4 " "	150 = 93
6 " "	130 = 80
Lake surface	100 = †62

* From Table 7, the evaporation from a 3-ft. pan = 100/80ths of that from a 6-ft. pan and 130% × 1.25 = 162 per cent.

† Identical with Professor Bigelow's value.

F. T. Robson, Assoc. M. Am. Soc. C. E., concludes,§ after a careful personal check and revision of the Salton Sea data, that the average evaporation depth from the Salton Sea during the 6 years, April 1st, 1907, to April 1st, 1913, was 67.02 in. per year. For the single year, June 1st, 1909, to June 1st, 1910, he accepts the evaporation depth from 6-ft. land-pans as 114.05 in. This value is the mean of the observed evaporation losses from four pans placed outside the "vapor blanket" but otherwise under the same desert conditions as the Salton Sea. These pans were north, east, and south of the sea, at distances of from 4 to 20 miles from its shore line. From these data, Mr. Robson estimates that the evaporation depth from the surface of the sea is $(67.02 \div 114.05) = 0.588$, or about 59% of that from 6-ft. square land-pans in that vicinity.

Since the evaporation from a 3-ft. pan. is $(100 \div 80) = 125\%$ of that from a 6-ft. pan, and the evaporation from a floating-pan is 80% of that from a land-pan of the same size, the evaporation from a 3-ft.

* "American Civil Engineers' Pocket Book" (Second Edition, 1912), p. 1256.

† Presumably a floating-pan.

‡ Presumably 2 ft. "square" instead of "in diameter".

§ *Transactions*, Am. Soc. C. E., Vol. LXXVI (December, 1913), pp. 1516–24.

square floating-pan will be (125% × 0.80) = 100%, the same as that from a 6-ft. square land-pan; hence, in Mr. Robson's opinion, the evaporation from the surface of the Salton Sea was about 59% of that from a 3-ft. square pan floating thereon.

C. E. Grunsky, M. Am. Soc. C. E., concludes,* from a careful personal revision of the Salton Sea data, that the evaporation from the sea during the year, April 1st, 1907, to April 1st, 1908, was 6.14 ft., or 73.7 in. Combining this value with the evaporation of the year, 1909-10, from 6-ft. square land-pans (though such a combination of two different years is not strictly permissible, and is not made by Mr. Grunsky), the evaporation from the Salton Sea itself is approximately (73.7 ÷ 114.05) = 64.6% of that from 6-ft. square land-pans situated near-by, or 3-ft. square pans floating thereon.

The several values found for the proportional evaporation from the Salton Sea itself and from 3-ft. square pans floating thereon are summarized in Table 14 (neglecting the 62% quoted from Merriman, which presumably is only Professor Bigelow's conclusion).

TABLE 14.—EVAPORATION FROM SALTON SEA, AS A PERCENTAGE OF THAT FROM 3-FT. SQUARE FLOATING-PANS.

Authority.	YEARLY EVAPORATION DEPTH, FROM:		Evaporation depth from Salton Sea, as a proportion of that from 3-ft. floating-pans.
	Salton Sea.	6-ft. land-pan or 3-ft. floating-pan.	
Bigelow	70 in.*§	*114.05 in.	61.4%
"	69 *§	"	60.5
Robson	67.02 †	"	58.8
Grunsky	73.7 ‡	"	64.6
Mean of all four values			61.3%
Mean of Bigelow's two values			60.95
Mean of Robson's and Grunsky's			61.7

* For the year, June 1st, 1909, to June 1st, 1910.

† Mean of 6 years, April 1st, 1907, to April 1st, 1913.

‡ For the year, April 1st, 1907, to April 1st, 1908.

§ Used by Professor Bigelow as 70 in.; stated by F. T. Robson as more accurately 69 in. from Professor Bigelow's data.

Mr. Robson states, as one of the conclusions of Professor Bigelow, that tests showed the pan evaporation from somewhat concentrated and brackish Salton Sea water to be 2% less than that from fresh water. However, the pan evaporations of Table 14 presumably are from fresh water, and the Salton Sea during 1907-13 presumably was very little brackish, and much less so than the concentrated sea water in the pans found to have 2% less evaporation than fresh water. Hence it is believed that the proportions in Table 14 (58.8%-64.6%)

* *Engineering News*, Vol. 60 (July–December, 1908), pp. 163–66.

do not require such 2% increases in applying them to reservoirs of fresh water elsewhere.

Of the four values in Table 14, only those of Professor Bigelow are derived from a comparison of the evaporation from the Salton Sea with that from 6-ft. land-pans or 3-ft. floating-pans for the same year; hence it is felt that Professor Bigelow's conclusions are more to be relied on than either Mr. Robson's or that derived from Mr. Grunsky's study.

For that reason, and also because Professor Bigelow's conclusion is based on a study of the values of the coefficient, C_2, his conclusion is that adopted. Hence his value of 62% (being in round numbers his 61.7% corresponding to his adopted value 0.024 of C_2) is that adopted and used throughout the remainder of this investigation.

The adopted conclusion of 62% appears to be based almost entirely on the experiments at the Salton Sea. Hence, for the specific purpose of this general investigation, a comparison seems necessary between the conditions of exposure, etc., existing at the Salton Sea and those at Lake Conchos.

The Salton Sea is regular and somewhat oval in form, and, at the time of the evaporation experiments, was about 45 miles long and from 10 to 16 miles broad, with its surface about 200 ft. below ocean or sea level. It had an area of about 430 sq. miles and about 115 miles of shore line or perimeter. Its maximum depth was about 75 ft. and its average depth about 30 ft. It lies in the trough of a shallow basin, and is open to the winds from every direction.

At spillway level, Lake Conchos will occupy a length, east and west, of about 24 miles, with a surface area of 67.7 sq. miles and 165 miles of shore line. Its shore lines are very irregular (as shown by Figs. 1 and 2), and it has a length on its broken easterly and westerly axis of approximately 30 miles. Its widths vary greatly: at spillway level from about $\frac{1}{2}$ mile to about $3\frac{3}{4}$ miles, with an average width of perhaps $2\frac{1}{4}$ miles. Its depths will be about 223 ft. maximum and about 61 ft. average below spillway level. At the average elevation of the lake surface during the full operation of the power-plant (1 312.5 m.), the average depth will be about 55 ft. and the maximum about 208 ft.; and during the 7 months from October, 1913, to April, 1914, they were about 32 and 131 ft., respectively.

As shown by Fig. 2 (a panoramic photograph), Lake Conchos occupies the somewhat canyon-like bed of the Rio Conchos, and in general its shores are steep and precipitous. Because of the adjacent hills, the steep shores, the many points projecting into the lake, and the consequent great variation in width, it is believed that the winds have much less free access to its surface than to that of the Salton Sea.

The principal reason for the smaller evaporation depth from large than from small water surfaces is believed to be the existence of

thicker and more unbroken "vapor blankets" over the larger surfaces, with less frequent changes of the overlying air in contact with the water by new dry and non-humid air. Also, because of its slower heating up, it is believed that there should be less evaporation depth from deep than from shallow reservoirs.

The quotient of a lake surface area divided by its perimeter or shore line is somewhat analogous to the "hydraulic radius" or semi-radius of a circular conduit; and, for comparative purposes, four times such quotients may be regarded as the equivalent mean diameters or widths of lakes. Such mean widths of the Salton Sea and of Lake Conchos, and also their average depths, are compared in Table 15.

TABLE 15.—COMPARATIVE MEAN WIDTHS AND MEAN DEPTHS OF SALTON SEA AND OF LAKE CONCHOS.

Lake and stage.	Area, in square miles.	Shore line, in miles.	Mean width, in miles.	Mean depth, in feet.
Salton Sea	430	115	15.0 = 100%	30 = 100%
Lake Conchos				
Elev. *1312.5 m.	55.3	152	1.46 = 9.7%	55 = 184%
" †1289. "	16.2	86.4	0.75 = 5.0%	32 = 107%

* Average lake level during future operation.

† Lake level during evaporation tests of October, 1913, to April, 1914.

Hence, during future operations, Lake Conchos will have about one-tenth as great a mean width and nearly twice as great a mean depth as that of Salton Sea during the evaporation tests at that sea; while, during the evaporation tests at Lake Conchos, its mean width was about one-twentieth of that of Salton Sea and its mean depth about the same.

With the information at hand, it is impossible to form any opinion of the comparative protective effects of their "vapor blankets" in these two instances. However, at least it is probable that the greater protection of Lake Conchos and its "vapor blanket" from the disturbing effect of winds because of its surrounding ridges, etc., will wholly or partly offset the less protection because of its smaller mean width and proportionally longer shore line. Also, the greater mean depth probably will tend to diminish the evaporation losses from Lake Conchos in the future, below their value during the 7 months, October, 1913, to April, 1914.

With regard only to differences in evaporation depths from large pans and from reservoirs and lakes, it seems probable, from Table 16 and a brief study of its values, on logarithmic co-ordinate paper, that the value of C_2 becomes, for practical purposes, constant and equal to 0.024 when pans reach a size as great as from 12 to 25 ft. square.

TABLE 16.—(FROM FIG. 5 AND TABLE 7.)

Size of pan.	Value of C_2.	Decrease in C_2 for 2 ft. increase in pan.
2 ft. square.....	0.042	
		0.006
4 " "	0.036	
		0.005
6 " "	0.031	
		0.003±
8 " "	0.028±	
		(0.004±)*
Lake surface.....	0.024	

* Decrease from 8-ft. pan to "lake surface".

From Table 16 and its logarithmic study, it appears at least very doubtful if there is any material variation in the values of C_2 as between smaller or larger large reservoirs and lakes.

Hence, in the absence of fuller information, the conclusion seemed justified that the relation established for the Salton Sea between the depth of evaporation from reservoirs and that from pans floating thereon, may be applied unchanged to Lake Conchos with a fair degree of confidence in its essential correctness.

Therefore, the yearly evaporation depth from the surface of Lake Conchos was adopted as 62% of the evaporation depth from a 3-ft. square pan floating thereon or from a 6-ft. square land-pan near-by.

Subsequently, in working up by Method *C* the evaporation depth from Lake Conchos, data were acquired permitting of a rough check from the evaporation tests at that lake of this adopted value of 62 per cent. This check is given in full hereinafter—and it will be stated here only that the value thus found at Lake Conchos was less than 67.5%, with a strong probability that it is as small as 62 per cent.

SUBSIDIARY CONCLUSION (*d*):

RELATIONS BETWEEN EVAPORATION DEPTH AND MEAN TEMPERATURE.

In studying the relations between evaporation and mean temperature, it will be necessary to consider in some detail the observed evaporations at each of the evaporation stations used in Texas and New Mexico; and, while doing this, occasion will be taken to discuss the accuracy, etc., of all such evaporation measurements, and to correct all the observed evaporations (from pans of various sizes) to the corresponding evaporations from a 3-ft. square floating-pan or a 6-ft. square land-pan.

The stations on the Great Plateau at which measurements of pan evaporation had been made are Austin and El Paso in Texas; and Albuquerque, Carlsbad, Elephant Butte, and Lake Avalon, in New Mexico. There also are available for temperature evaporation studies, Piche evaporimeter records at eleven stations.

TABLE 17.—Measured Monthly

Station.	Abilene, Tex.		Austin, Tex.		Brownsville, Tex.		Corpus Christi, Tex.		El Paso, Tex.		El Paso, Tex. (at Ft. Bliss.)		El Paso, Tex. (at Ft. Bliss.)		Fort Davis, Tex.		Ga
Elevation, in feet.	1 740 ±		600 ±		50 ±		20 ±		3 762		3 700±		3 700±		4 980 ±		
Character.	Floating-pan.*		Floating-pan, 30 in.		Floating-pan.*		Floating-pan.*		Floating-pan.*		Floating-pan, 3 ft.		Floating-pan, 3 ft.		Floating-pan.*		F
Period observed.	July, 1887–June, 1888.		January-December, 1911.		July, 1887–June, 1888.		July, 1887–June, 1888.		July, 1887–June, 1888.		June, 1889–May, 1890.		Sept., 1889–August, 1890.		July, 1887–June, 1888.		July
Months.	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evapor in inc
January	1.8	38.9	2.52	56.5	1.8	55.0	1.4	48.3	4.0	44.8	⊙ 2.0	48.3	⊙ 2.0	48.3	5.4	45.0	1.
February	1.7	48.0	1.72	58.8	2.6	65.3	1.6	60.3	3.9	50.1	2.0	51.8	2.0	51.8	5.7	48.2	2.
March	3.1	49.8	2.62	64.0	2.9	66.4	3.8	61.9	6.0	54.6	7.0	58.8	7.0	58.8	6.7	50.1	3.
April	4.2	65.4	3.07	67.0	3.0	74.2	5.0	72.6	8.4	66.9	7.3	64.3	7.3	64.3	8.5	61.5	3.
May	5.0	69.6	4.70	73.3	3.5	78.0	3.3	76.6	10.7	71.4	10.8	75.2	10.8	75.2	11.0	66.8	4.
June	5.8	77.8	7.96	84.2	3.9	80.9	3.9	80.7	13.6	83.0	10.7	80.8	11.7	79.6	12.0	76.1	4.
July	9.5	83.9	7.33	84.2	4.0	81.2	4.4	82.9	9.4	80.9	9.6	82.0	9.6	82.3	11.4	75.4	5.
August	7.5	81.3	8.58	86.3	4.1	81.8	4.3	83.0	7.7	79.9	11.4	82.7	7.6	78.8	9.0	74.6	5.
September	6.2	75.9	6.69	84.2	3.3	77.9	4.8	79.7	5.6	74.3	9.2	71.4	9.2	71.4	8.9	68.3	5.
October	4.5	61.7	2.86	67.8	3.0	70.0	4.1	69.4	5.2	63.4	6.8	65.0	6.8	65.0	5.2	57.7	4.
November	3.4	52.6	1.51	55.0	2.6	66.3	3.0	64.0	4.6	54.3	4.6	49.0	4.6	49.0	5.7	53.3	4.
December	1.7	40.3	1.36	49.0	2.3	56.5	2.3	53.3	2.9	40.2	2.9	53.2	2.9	53.2	4.9	40.0	2.
Total or mean	54.4	63.1	50.92	69.2	37.0	71.0	38.8	69.4	82.0	63.6	84.3	65.3	81.5	64.3	91.4	59.7	46.
References	Turneaure and Russell, p. 60.	U. S. W. B.	W. S. & I. No. 308, p. 17.	U. S. W. B.	T. & R. p. 59.	U. S. W. B.	T. & R. p. 58.	U. S. W. B.	T. & R. p. 60.	U. S. W. B.	12th Annual Report, U. S. G. S. Pt. 2, p. 235.	U. S. W. B.	12th Annual Report, U. S. G. S. Pt. 2, p. 235.	U. S. W. B.	T. & R. p. 60.	U. S. W. B.	T. & p. 5
Notes.	*Based on Piche evaporimeter calculations, said to be equivalent to floating-pan records.		Careful daily record on 30-in. pan floating in cistern.		*See *re* Abilene.		*See *re* Abilene.		*See *re* Abilene.		The following additional record later discovered in 14th Annual Report, U. S. G. S., Pt. 2, p. 154: January, 1892..... 2.4 in..... 43° February, " ..... 3.2 " ..... 50° March, " ..... 6.0 " ..... 55° April, " ..... 7.5 " ..... 64° May, " ..... 10.0 " ..... 71° June, " ..... 13.0 " ..... 81° July, " ..... 12.5 " ..... 83° August, " ..... 11.9 " ..... 79° January, 1893..... 4.3 " ..... 46° February, " ..... 4.8 " ..... 50° March, " ..... 4.0 " ..... 55° April, " ..... 9.3 " ..... 66°				*See *re* Abilene.		*

⊙ *re* El Paso (Ft. Bliss): All monthly depths of evaporation are measured for more than 20 days, but no month was completely measured. The depths given are the average daily for the period measured, multiplied by applies also to records of 1892 and 1893, except that January and August, 1892, and March, 1893, were measured for less than 10 days, and April, 1893, for less than 20 days.

TABLE 17.—Measured Monthly Evaporations and Corresponding Mean Monthly Temperatures‡ for Stations in Texas and New Mexico.

El Paso, Tex. (at Ft. Bliss.)		Fort Davis, Tex.		Galveston, Tex.		Palestine, Tex.		Rio Grande City, Tex.†		San Antonio, Tex.		Albuquerque, N. Mex.		Albuquerque, N. Mex.	
3 700±		4 920 ±		6 ±		500 ±		230 ±		700 ±		5 000 ±		5 000 ±	
Floating-pan, 3 ft.		Floating-pan.*		Floating-pan.*		Floating-pan.*		Floating-pan.*		Floating-pan.*		Land-pan, 2 ft.		Land-pan, 2 ft.	
pt., 1889–August, 1890.		July, 1887–June, 1888.		July, 1887–June, 1888.		July, 1887–June, 1888.		July, 1887–June, 1888.		July, 1887–June, 1888.		Feb., 1900–Jan., 1901.		Jan.–Dec.,* 1903.	
poration, inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthl temperat (F°).
⊙															
2.0	48.3	5.4	45.0	1.6	49.8	2.1	43.4	2.7	54.7	2.4	48.0	2.04	35.4	1.81	36.3
2.0	51.8	5.7	48.2	2.8	58.8	3.0	52.3	3.5	63.6	3.3	56.5	2.63	40.6	2.07	34.6
7.0	58.8	6.7	50.1	3.2	60.1	3.3	53.3	3.5	64.2	4.1	56.6	6.17	49.1	5.31	47.2
7.3	64.3	8.5	61.5	2.9	71.2	4.2	68.5	3.6	74.5	3.8	69.3	6.82	52.0	10.05	56.5
10.8	75.2	11.0	66.8	4.3	74.8	4.3	70.4	4.5	78.6	4.0	72.8	10.08	67.0	10.98	62.4
11.7	79.6	12.0	76.1	4.2	80.2	4.5	77.7	4.6	81.2	4.5	78.0	12.63	76.3	11.33	68.4
9.6	82.3	11.4	75.4	5.3	81.9	5.8	82.0	6.9	84.5	6.6	83.8	11.78	79.1	12.36	75.2
7.6	78.8	9.0	74.6	5.2	82.2	4.6	80.9	7.0	85.4	5.8	83.0	10.21	75.0	11.78	74.2
9.2	71.4	5.9	68.3	5.2	79.4	4.8	75.2	5.2	80.2	5.2	77.5	8.00	67.7	9.65	64.0
6.8	65.0	5.2	57.7	4.7	70.0	4.4	63.9	4.9	70.2	5.4	66.9	4.88	56.8	6.62	53.9
4.6	49.0	5.7	52.3	4.2	63.0	4.0	56.5	3.6	65.1	4.2	59.5	1.73	44.7	4.21	42.4
2.9	53.2	4.9	40.0	2.4	59.6	2.1	45.6	3.1	54.2	3.1	49.4	1.40	34.6	1.88	38.7
81.5	64.8	91.4	59.7	46.0	68.7	47.1	64.1	53.1	71.4	52.4	66.8	77.87	56.5	87.90	54.1
n Annual Report, S. G. S. 2, p. 235.	U. S. W. B.	T. & R. p. 60.	U. S. W. B.	T. & R. p. 58.	U. S. W. B.	T. & R. p. 58.	U. S. W. B.	T. & R. p. 59.	U. S. W. B.†	T. & R. p. 58.	U. S. W. B.	W. S. & I. No. 188, p. 31.	U. S. W. B.	W. S. & I. No. 188, p. 31.	U. S. W.

ecord later discovered
. S., Pt. 2, p. 154:
.4 in..... 48°
.2 " ..... 50°
.0 " ..... 55°
.5 " ..... 64°
.0 " ..... 71°
.0 " ..... 81°
.5 " ..... 88°
.9 " ..... 79°
.3 " ..... 46°
.8 " ..... 50°
.0 " ..... 55°
.2 " ..... 66°

* See *re* Abilene.

* See *re* Abilene.

* See *re* Abilene.

* See *re* Abilene.
† Ft. Ringgold.

* See *re* Abilene.

Experiments at Univ. N. Mex., published in *B*
No. 10, Vol. 3, Hadley Climatological Lab., '05.
* This record continued as follows:
January, 1904..... 2.28 in., 36.4°
February, " ..... 3.92 " 50.8°
March, " ..... 7.48 " 57.2°
April, " ..... 10.71 " 58.2°
May, " ..... 12.53 " 65.0°
June, " ..... 13.03 " 71.3°

the average daily for the period measured, multiplied by the number of days in the month. This
ys.

‡ The mean monthly temperatures here used are the means of the daily mean temperatures, v
the maximum and minimum each day.

N. Mex.	Carlsbad, N. Mex.		Carlsbad, N. Mex		Elephant Butte, N. Mex.		Ft. Stanton, N. Mex.		Lake Avalon, Carlsbad, N. Mex.		Santa Fé, N. Mex.		Station.
:	3 000 ±		3 000 ±		4 250 ±		6 150 ±		3 200 ±		7 013		Elevation, in feet.
3 ft.	Land-pan, 3 ft.		Land-pan, 3 ft.		Land-pan, 4 ft.		Floating-pan.*		Floating-pan, 4 ft.		Floating-pan.*		Character.
1903.	Jan.-Dec., 1899.		Jan.-Dec., 1901.		July, 1909-June, 1910.		July, 1887-June, 1888.		July, 1909-June, 1910.		July, 1887-June, 1888.		Period observed.
Mean monthly nperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Evaporation, in inches.	Mean monthly temperature (F°).	Months.
36.3	1.26*	42.2‡	1.04*	44.7¶	2.5 *	42.5	3.9	36.3	4.50*	46.4	3.0	30.3	January.
34.6	1.28*	42.8‡	1.04*	47.0¶	2.75*	44.2	3.9	38.0	4.50*	47.6	3.4	35.5	February.
47.2	3.14*	56.4‡	2.57*	56.9¶	4.50*	57.2	5.2	38.9	5.51	61.3	4.2	37.4	March.
56.5	3.79*	63.3‡	1.75	61.1§	8.00*	61.3	7.3	52.8	7.45	64.4	6.8	49.9	April.
62.4	5.58§	72.0‡	5.00	71.9¶	11.50*	71.2	9.5	56.5	10.12	72.3	8.8	53.4	May.
68.4	9.01§	78.2‡	6.75	81.4‡	13.45	78.8	10.9	67.9	11.05	81.2	12.9	67.1	June.
75.2	8.02	80.6‡	6.50	80.2‡	11.57	81.1	9.4	65.6	13.88	83.0	9.2	66.9	July.
74.3	7.90	83.6‡	7.25	81.6‡	10.48	79.2	11.6	63.5	13.00*	79.0	9.8	65.8	August.
64.0	5.60	74.5‡	4.75	74.1§	8.58	70.7	8.9	60.1	9.50*	71.8	6.6	60.8	September.
53.9	4.20	63.6‡	3.75	66.6‡	6.76	61.0	4.0	48.5	7.00*	69.2	6.7	50.6	October.
42.4	3.23*	53.4¶	1.72*	56.6‡	3.86	51.8	3.6	41.6	5.75*	57.2	5.7	41.9	November.
38.7	1.39*	43.2¶	1.14*	46.8‡	3.00*	34.4	3.8	30.0	4.50*	39.6	2.7	26.8	December.
54.1	54.37	68.1	43.26	64.1	86.95	61.1	77.0	50.0	94.51	63.8	79.8	43.9	Total or mean.
. S. W. B.	O. E. S. *Bul. No. 86*, p. 109.	‡U. S. W. B. ‖O. E. S., *Bul. No. 86*, p. 109. ¶Long period mean, U. S. W. B.	O. E. S. *Bul. No. 119*, p. 42.	‡U. S. W. B. §O. E. S., *Bul. No. 119*, p. 42. ‖Long period mean, U. S. W. B.	*Eng. News*, v. 63, p. 695.	U. S. W. B.	T. & R. p. 60.	U. S. W. B.	*Eng. News*, v. 63, p. 695.	U. S. W. B.	T. & R. p. 60.	U. S. W. B.	References.
d in *Bull.* '05.	*Deduced from monthly percentage distribution relations. §These records are believed to be correct. Original text gives greater values, probably due to a confusion of cumulative evaporation with weekly depths.		*Deduced from monthly percentage distribution relations.		Rio Grande Dam, 10 miles west of Engle, N. Mex. *Deduced.		* See *re* Abilene.		*Deduced.		*See *re* Abilene.		Notes.

ures, which are, in turn, means between

A study of these data (all of which are given in Table 17, Plate LXII) develops the fact that for each station there exists a stable average relation between its monthly depth of evaporation and its monthly mean temperature; and that on the Great Plateau the extreme relations between evaporation and temperature depart so little from the average that (where the mean temperatures are known at two places and the evaporation at only one of them) safe use may be made of this average relation to estimate the evaporation at the other.

It is well known that many other conditions besides temperature affect the depth of evaporation, such as prevalence and velocity of the wind, humidity, vapor tension, barometric conditions, etc. However, these other conditions are almost entirely unknown in most cases where engineers are called on to estimate the probable annual evaporation—whereas mean temperatures frequently are of record. It is fortunate, therefore, that the joint effect of all the climatic conditions except mean temperature is so small (either because of the relative unimportance of each, or because some tend to counteract others) that a direct comparison of mean temperature with evaporation depth at any station on the Great Plateau leads to an average relation so stable and so essentially accurate that its use is justifiable in estimating evaporation depths, even when unaided by a knowledge of the other climatic conditions which affect evaporation.

That these relations between mean monthly temperature and evaporation are stable and consistent is shown by the diagrams which have been prepared for each record station and which are given in the following discussion.

Austin, Tex.—The record of pan evaporation at Austin appears to be an excellent one. The evaporation record for an entire year of daily observation has been published,* the evaporation being that from a floating-pan 30 in. square.

From these data and from temperatures given in the records of the U. S. Weather Bureau (all as summarized in Table 17, Plate LXII) is drawn the heavy line of Fig. 6, which shows the average relations between the monthly mean temperatures and the inches of depth of measured monthly evaporations at Austin for a floating-pan 30 in. square. The light line of Fig. 6 (derived from the heavy line by means of the relations shown by Table 2 and Fig. 5) gives those relations for a 3-ft. square floating-pan.

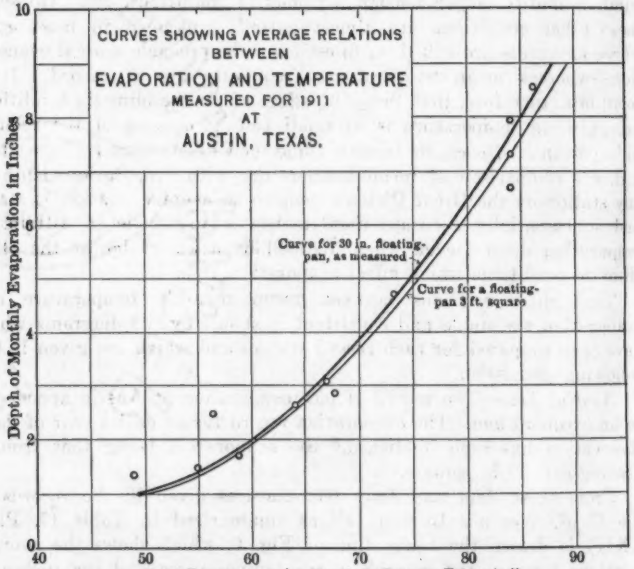
The regularity and consistency of the curves of Fig. 6 leave no doubt (at least for Austin, Tex.) of the existence of a regular relation or law between evaporation depths and mean temperatures.

El Paso, Tex.—The record at El Paso comprises evaporations as measured in 3-ft. square floating-pans for 34 months in 1889-90 to 1892-93; and 12 months Piche evaporimeter measurements in 1887-88.

* *Water Supply and Irrigation Bulletin No. 308*, p. 17.

The sources of all the El Paso data are stated in connection with their summary in Table 17 (Plate LXII).

The record at El Paso is of much importance, as it is the nearest to Lake Conchos of any of the measurements of evaporation made in Texas and New Mexico. It happens, however, that there were many breaks in the observations, from causes not stated, that no single month's record is complete, and that the full monthly evaporations given in Table 17 (Plate LXII) necessarily were obtained only by multiplying the average daily evaporation for the days observed each month by the number of days in the full month.



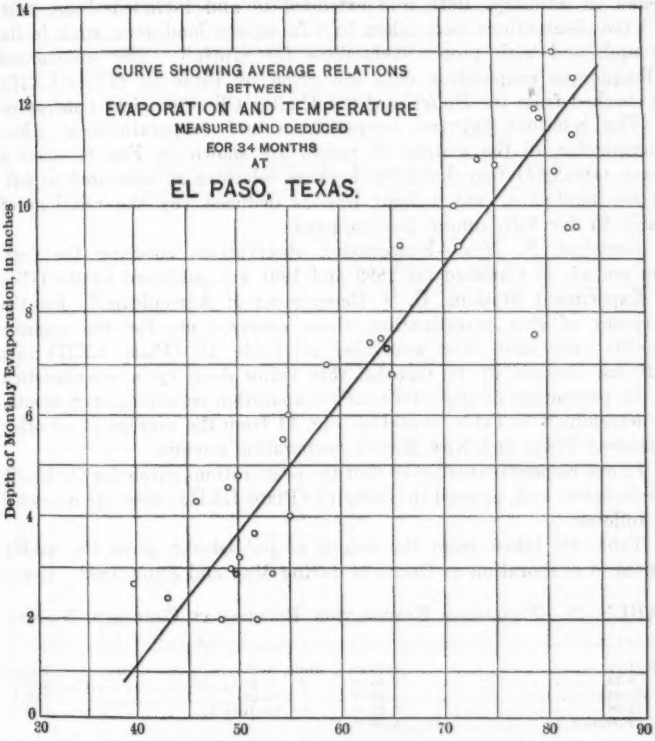
NOTE:- This record of evaporation was measured in a 30 in. floating-pan for the year 1911. The pan was at Elevation 600 ft. at Austin, Tex. For details of record see Table 17. The total measured evaporation for the year 1911, as plotted above, was (for 30-in.pan) 50.92 in., and by the Average Curve shown above(for 30 in.pan) is 51.4 in.

FIG. 6.

The relations between mean monthly temperatures and depth of monthly evaporation at El Paso are shown by the curve (in this case a straight line) of Fig. 7, which is plotted from the El Paso data of Table 17 (Plate LXII). As for Austin (and for all the other stations studied), the curve indicates clearly a regular relation or law between temperature and depth of evaporation.

Albuquerque, N. Mex.—The published record of the evaporation at Albuquerque was found first in the Water Supply and Irrigation

Papers,* but very few data were there given as to how the observations were made. However, after much search and correspondence, there was obtained from the University of New Mexico a copy of one of the



NOTE: This record of evaporation was measured in a floating-pan, 3 ft. square, for the following periods,- May, '89 to Aug. '90; Nov. 90 to Apr., 91; Jan. to Aug., 92; and Jan. to Apr., '93. The pan was near Fort Bliss, at El Paso, Texas, at Elevation 3700 ft. ±. The measurements were in no case continuous for a full month, but in general covered more than 20 days. For details of the record see Table 17. The total depth of evaporation during the 34 months of record was 230.10 in. and the corresponding total by the Average Curve shown above is 240.26 in.

FIG. 7.

Bulletins of the Hadley Climatological Laboratory of that University,† from which the facts concerning the Albuquerque evaporation observations were ascertained to be as follows:

* *Bulletin No. 188*, p. 31.
† No. 10, Vol. III, 1905.

The observations were taken from February, 1900, to and including January, 1901, instead of for the calendar year 1900, as given in the Water Supply and Irrigation Bulletin. The second set of observations began in January, 1903, and extended to and included June, 1904. All the observations were taken in 2-ft. square land-pans, sunk in the ground, and with proper deductions for rainfall. The summarized Albuquerque evaporation data are given in Table 17 (Plate LXII), as checked from the *Bulletin* of the Hadley Climatological Laboratory.

The relations between temperatures and evaporations at Albuquerque for all the months of record are shown on Fig. 8, where a heavy (straight) line shows the average relations as measured in 2-ft. square land-pans, and a light line as deduced (by the relations of Table 2) for 3-ft. square floating-pans.

Carlsbad, N. Mex.—Evaporation observations covering the summer periods at Carlsbad for 1899 and 1901 are published by the Office of Experiment Stations, U. S. Department of Agriculture.* For the purposes of this investigation, these evaporations for the summer months only have been expanded in Table 17 (Plate LXII) into full-year records of 12 months, this being done by a consideration of the percentage of the total year's evaporation occurring each month, as determined in Table 20 and on Fig. 11 from the average of all other measured Texas and New Mexico evaporation records.

In one respect it is believed that the evaporations given for Carlsbad† are in error, and, as used in Table 17 (Plate LXII), they are corrected as follows:

Table 18, taken from the record as published,‡ gives the weekly depths of evaporation at Carlsbad during May and June, 1899. Hence

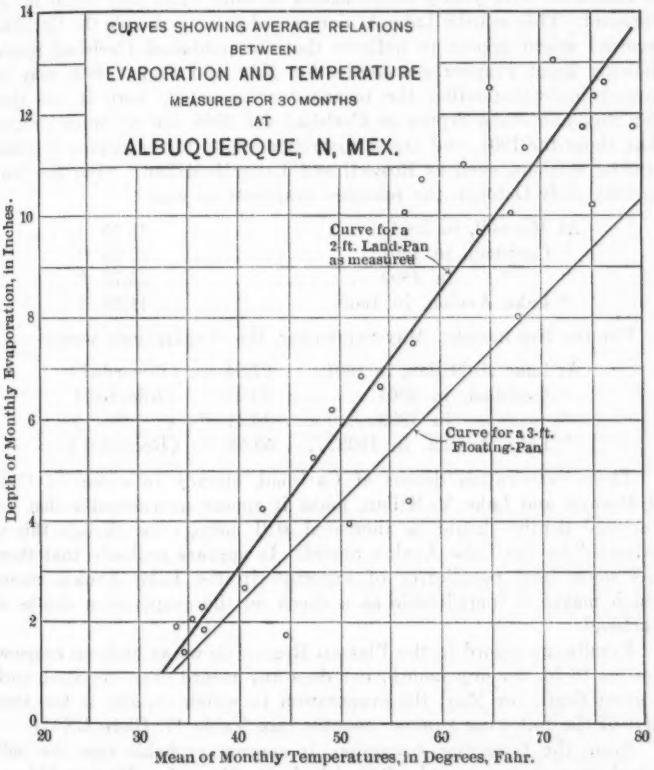
TABLE 18.—PUBLISHED EVAPORATION RECORDS AT CARLSBAD, N. MEX.

May 1–7	1.13 in.	June 4–10	2.50 in.
" 8–14	2.50 "	" 11–17	2.85 "
" 15–21	4.25 "	" 18–24	2.00 "
" 22–28	5.25 "	" 25–July 1	1.68 "
" 29–June 3	5.88 "		
Total	19.01 in.	Total	8.98 in.
Say, less 5.88 × 5/6 = 2.94 in. for June 1–3	−2.94 "	Say, plus 2.94 in. less 1.63 × 1/7 = 0.23	+2.71 "
For May, say	16.07 in.	For June, say	11.69 in.

the published records indicate that the evaporation at Carlsbad was about 16 in. for May, and nearly 12 in. for June, 1899.

* *Bulletins Nos. 86* and *119*.
† *Bulletin No. 86*, p. 109.
‡ In *Bulletin No. 86*.

Such great monthly evaporations seem to be very improbable, as Carlsbad is not a station having a record (except for the 2 months, May and June, 1899) which shows excessive monthly evaporation depths, and as the depths for May and June, 1901, were only 5.00 and



NOTE:-This record of evaporation was measured in a 2-ft. Land-Pan, during the periods from Feb., 1900, to Jan., 1901, incl., and Jan., 1903, to June, 1904, incl. For details of the record see Table 17.

The total evaporation during the 30 months of measurement was 215.72 in. (for 2-ft. pan) and for the same months by the Average Curve above (for 2-ft. pan) 218.54 in.

FIG. 8.

6.75 in., respectively, and not materially different from the depths for July and August. Also, the evaporation depths for May and June, 1901, at Carlsbad appear to be reasonable because of the similar depths at Lake McMillan, not far distant.

Another cause for suspicion of the correctness of the published record at Carlsbad is the much greater evaporation depth observed at Lake Avalon, Carlsbad (See Table 17, Plate LXII). The Lake Avalon record gives 94.51 in. for 12 months' evaporation in 1909-10, as compared with yearly evaporations of only 54.37 and 43.26 in. at Carlsbad. This near-by Lake Avalon record (almost double the Carlsbad records) might appear to indicate that the published Carlsbad record showing great evaporation depths for May and June, 1899, may be correct, notwithstanding the inconsistencies noted, were it not that the May and June depths at Carlsbad for 1899 are so much greater than those for 1901, and are so disproportionate to the depths of other near-by stations, such as Roswell and Lake McMillan.* For the four months, July-October, the recorded evaporations were:

At Roswell, in 1901	15.55 in.
" Carlsbad, in 1901	22.25 "
" " in 1899	25.72 "
" Lake Avalon, in 1909	41.38 "

For the five months, May-September, the evaporations were:

At Lake McMillan, in 1901	27.84 in.	(Recorded.)
" Carlsbad, in 1901	30.25 "	(Adopted.)
" " in 1899	36.11 "	(")
" Lake Avalon, in 1909	55.55 "	(Recorded.)

These evaporation depths at Carlsbad, already in excess of those at Roswell and Lake McMillan, make it appear unreasonable that the Carlsbad depths should be increased still more, even though this is indicated by the Lake Avalon record. It appears probable that there was some local peculiarity of exposure in the Lake Avalon record which makes it inapplicable as a check on the evaporation depths at Carlsbad.

Finally, no record in the Plateau Region shows as high an evaporation as 16 in. for any month, nor does any record even approach such a great depth for May, the evaporation in which usually is less than that of the following summer months (see Table 17, Plate LXII).

From the foregoing discussion, it appears probable that the published evaporation record of Carlsbad, N. Mex., for May and June, 1899, is in error, and gives depths which are excessive.

In an attempt to correct the record, a study of the weekly evaporations for May and June, 1899, as published, makes it appear probable that the error was in using the weekly evaporations between May 7th and June 3d cumulatively instead of separately, probably by a clerical error.†. On this assumption, the published records are corrected as in Table 19.

* *Bulletin No. 119*, Office of Experiment Stations, p. 42.

† See note under Table 19.

TABLE 19.—CORRECTED RECORDS FOR CARLSBAD, N. MEX.

Dates.	Published.*	Corrected.
May 1- 7	1.13 in.	1.13 in.
" 8-14	2.50 "	1.37 "
" 15-21	4.25 "	1.75 "
" 22-28	5.25 "	1.00 "
" 29-June 3	5.88 "	0.63 "
Totals	19.01 in.	5.88 in.
June 4-10	2.50 in.	2.50 in.
" 11-17	2.85 "	2.85 "
" 18-24	2.00 "	2.00 "
" 25-July 1	1.63 "	1.63 "
Totals	8.98 in.	8.98 in.

May 1–June 3 (corrected)	5.88 in.
less June 1–3 = 0.63 × 3/5 (?), say	−0.31
Corrected evaporation for May, 1899	5.57 in.
June 4–July 1 (published and corrected)	8.98 in.
Plus June 1–3, 0.30 in., and less July 1, 1.63 × 1/7 = 0.23 in.	+0.07 in.
Corrected for June, 1899	9.05 in.

* It is noticeable that the weekly evaporation increases rapidly and continuously up to June 3d, and then drops suddenly. (By some error in the computation, the values obtained and used in the investigation were 5.58 in. for May and 9.01 in. for June.)

The average relations between temperatures and evaporations at Carlsbad are shown by the diagram, Fig. 9, where the months actually observed are shown by heavy circles, those only deduced by light circles, the observed relations for 3-ft. square land-pans by a heavy curve, and the deduced relations for 3-ft. square floating-pans (assumed as 80% of the land-pans) by a light curve.

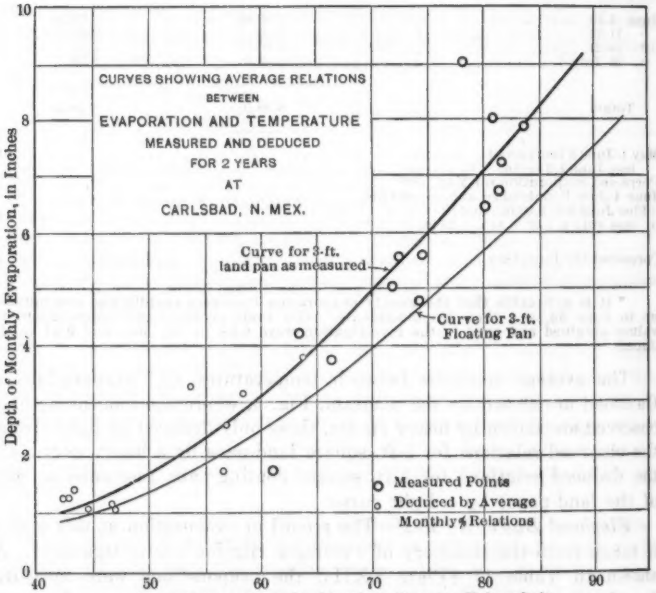
Elephant Butte, N. Mex.—The record of evaporation at this station is taken from the summary of Professor Bigelow's investigation.* As shown in Table 17 (Plate LXII), the evaporations were measured for the 6 months, June-November, 1909, in a 4-ft. square land-pan. The evaporations of the 6 months, December-May, were deduced by some method not well described. The deductions appear to have been made properly, however, from the fact that the deduced values for the missing months show the same percentage relations to the entire yearly evaporation as those shown on Fig. 11 and in Table 20 for the average of all the measured evaporation stations.

The temperature evaporation relations for Elephant Butte are shown by diagram on Fig. 10, where the observed and deduced values are distinguished, and the relations are shown for both a 4-ft. square land-pan and a 3-ft. square floating-pan.

* As given in *Engineering News*, Vol. 63, pp. 694–695.

Because of its comparative nearness, that at Elephant Butte is perhaps the most important (next to that at El Paso) of any of the Texas and New Mexico evaporation records available for the determination of the probable evaporation at Lake Conchos.

Lake Avalon, N. Mex.—This evaporation record, also, is taken from Professor Bigelow's investigation.* As shown there and in Table 17 (Plate LXII), the Lake Avalon evaporations were measured only for the 5 months, March-July, 1909, while those for the 7 months, August-February, were deduced by some method not clearly stated.



NOTE:- This record of evaporation was measured in a 3-ft. land-pan during 1899 and 1901. The elevation of Carlsbad is 3000 ± ft. The details of this record, part of which is deduced, are given on Table 17.

The total evaporation during the 13 months of actual measurement was (for land-pan) 76.07 in. and for the same months by the Average Curve shown above (for land-pan) is 77.51 in.

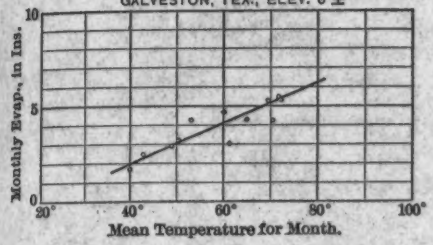
FIG. 9.

As pointed out in the discussion of the Carlsbad records, there is a disagreement between the Lake Avalon records and those for land-pans at Carlsbad. The latter are checked both for evaporation depth and for percentage distribution among the months by the independent records at the near-by stations of Roswell and Lake McMillan; and as

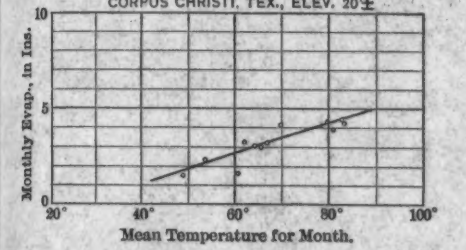
* *Engineering News*, Vol. 63, pp. 694–695.

CURVES SHOWING RELATIONS BETWEEN
EVAPORATION AND TEMPERATURE
AS DETERMINED BY
PICHE EVAPORIMETER RECORDS
IN
TEXAS AND NEW MEXICO
1887-1888.

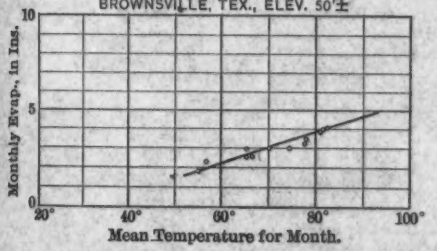
GALVESTON, TEX., ELEV. 6'±



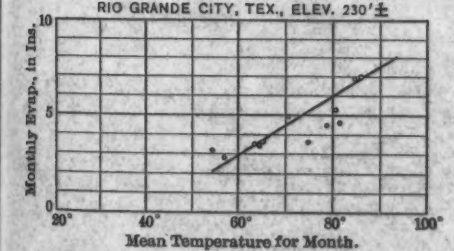
CORPUS CHRISTI, TEX., ELEV. 20'±



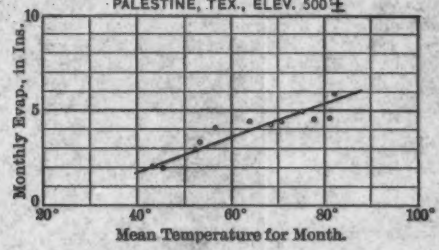
BROWNSVILLE, TEX., ELEV. 50'±



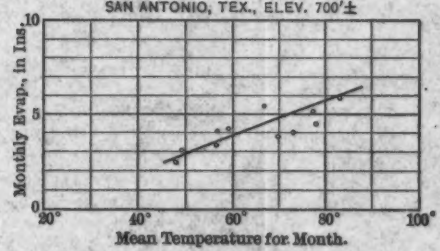
RIO GRANDE CITY, TEX., ELEV. 230'±



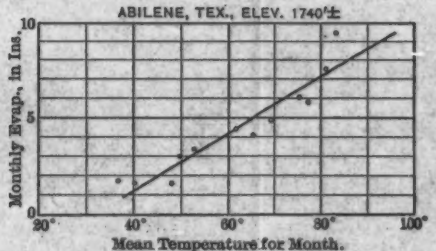
PALESTINE, TEX., ELEV. 500'±



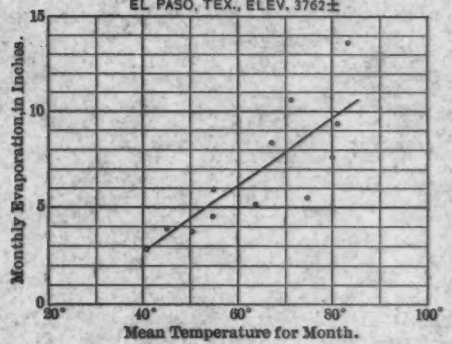
SAN ANTONIO, TEX., ELEV. 700'±



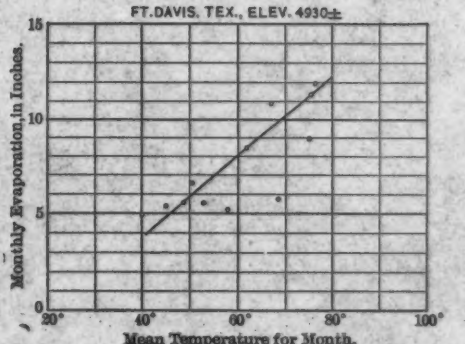
ABILENE, TEX., ELEV. 1740'±



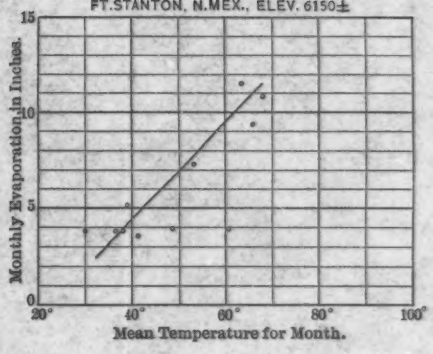
EL PASO, TEX., ELEV. 3762±



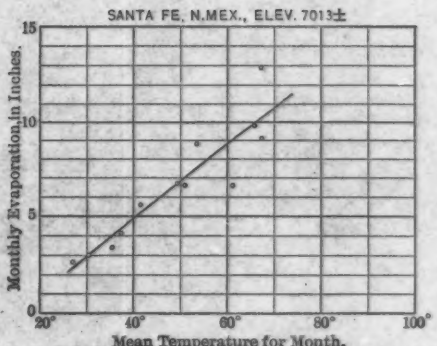
FT. DAVIS, TEX., ELEV. 4930±



FT. STANTON, N. MEX., ELEV. 6150±



SANTA FE, N. MEX., ELEV. 7013±



CURVES SHOWING RELATIONS BETWEEN
EVAPORATION AND TEMPERATURE
AS DETERMINED BY
PICHE EVAPORIMETER RECORDS
IN
TEXAS AND NEW MEXICO
1887-1888.

ELEV. 500'±

80° 100°

re for Month.

., ELEV. 700'±

80° 100°

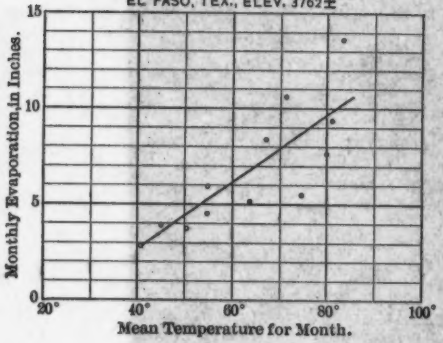
re for Month.

ELEV. 1740'±

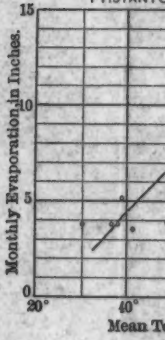
80° 100°

re for Month.

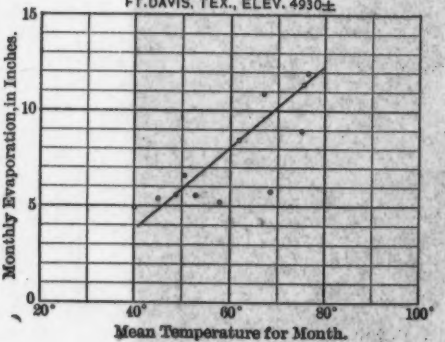
EL PASO, TEX., ELEV. 3762±



FT. STANTO



FT. DAVIS, TEX., ELEV. 4930±



SANTA F

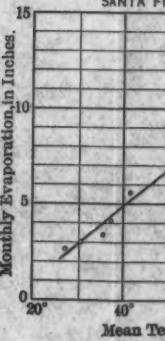
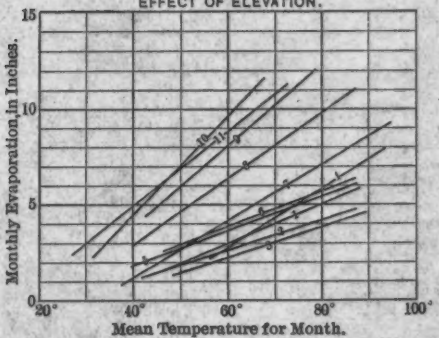


PLATE LXIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXX, No. 1376.
DURYEA AND HAEHL ON
EVAPORATION FROM LAKE CONCHOS, MEXICO.

COMBINED
TEMPERATURE-EVAPORATION CURVES
SHOWING
EFFECT OF ELEVATION.



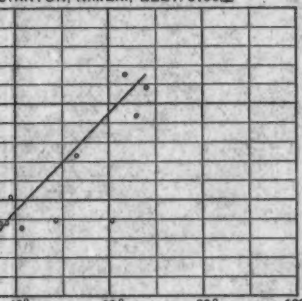
The above curves are numbered in the order of elevation above Sea Level as follows:

Curve 1, elev. 6; Curve 5, elev. 500; Curve 9, elev. 4930;
Curve 2, " 20; Curve 6, " 700; Curve 10, " 6150;
Curve 3, " 50; Curve 7, " 1740; Curve 11, " 7013;
Curve 4, " 230; Curve 8, " 3762;

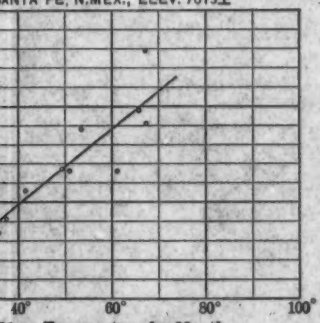
Notes.-All Temperatures are in Degrees, Fahrenheit.

The records of evaporation plotted hereon are from calculations by the United States Signal Service, based on Piche Evaporimeter records for 1887-88. For the detailed records see Table 17. The depths of evaporation here given are said to be equivalent to "water surface" evaporation, and probably correspond fairly to floating-pan records. Subsequent measurements in pans, at scattered stations in the United States, are said to indicate that the Piche records give somewhat greater depths of evaporation than those measured.

STANTON, N.MEX., ELEV. 6150±

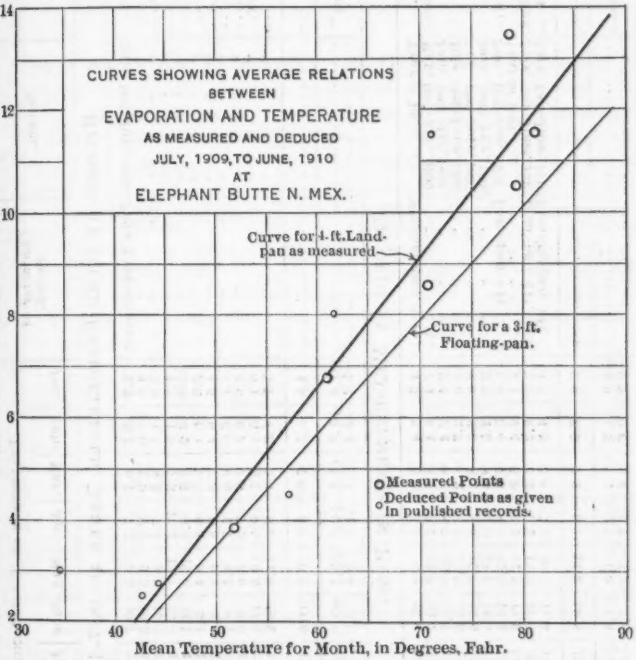


SANTA FE, N.MEX., ELEV. 7013±



Lake Avalon is near Carlsbad, an attempt is made here to analyze the disagreement between their records.

As shown in Table 17 (Plate LXII), the total year's evaporation at Lake Avalon was 94.51 in., as published. If the evaporation depths for the 7 missing months are deduced by use of the mean monthly percentage distributions for all measured evaporation stations on the



NOTES:- This record of evaporation was measured in a 4-ft. Land-pan for the season July, 1909, to June, 1910, incl. The pan was at Elephant Butte, on the Rio Grande River, near Engle, N. Mex., at Elevation 4250 ft. ±. The details of the record are given on Table 17.

A portion of the season's record was deduced, but the published record does not state the manner in which this was done.

The total evaporation during the months of actual measurement was (for 4-ft. pan) 54.7 in. and for the same months by the Average Curve shown above (for 4-ft. pan) 55.6 in.

FIG. 10.

Plateau (see Fig. 11 and Table 20), the year's evaporation depth would be reduced to 85.8 in. In Fig. 12, the evaporation temperature curve for Lake Avalon (a straight line) is drawn by considering the points of actual measurement only, neglecting the seven deduced points; and if the depths for the missing 7 months are supplied from that diagram, the total year's evaporation will be approximately 87 in.

TABLE 20.—PERCENTAGE DISTRIBUTION OF EVAPORATION BY MONTHS FOR STATIONS IN TEXAS AND NEW MEXICO.

Station.	Season.	Character of record.	PERCENTAGE OF TOTAL ANNUAL EVAPORATION, BY MONTHS.												
			Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Total.
RECORDS BY PICHE EVAPORIMETER, TAKEN IN 1887–1888.															
Abilene, Tex.	July, 1887-June, 1888.	Piche Evaporimeter.	3.3	3.1	5.7	7.7	9.2	10.7	17.5	13.8	11.4	8.3	6.2	3.1	100
Brownsville, Tex.	" " – " "	" "	4.9	7.0	7.9	8.1	9.5	10.5	10.8	11.1	8.9	8.1	7.0	6.2	100
Corpus Christi, Tex.	" " – " "	" "	3.6	4.1	8.5	7.7	8.3	10.1	11.3	11.1	11.1	10.6	7.7	5.9	100
El Paso, Tex.	" " – " "	" "	4.9	4.8	7.3	10.2	13.1	16.6	11.5	9.4	6.8	6.3	5.6	3.5	100
Fort Davis, Tex.	" " – " "	" "	5.9	6.2	7.3	9.3	12.0	13.1	12.5	9.9	6.5	5.7	6.2	5.4	100
Galveston, Tex.	" " – " "	" "	3.5	6.1	7.0	6.3	9.4	9.1	11.5	11.3	11.3	10.2	9.1	5.2	100
Palestine, Tex.	" " – " "	" "	4.5	6.4	7.0	8.9	9.1	9.6	12.2	9.8	10.2	9.3	8.5	4.5	100
Rio Grande, Tex.	" " – " "	" "	5.1	6.6	6.6	6.7	8.5	8.7	13.0	13.2	9.8	9.2	6.8	5.8	100
San Antonio, Tex.	" " – " "	" "	4.6	6.3	7.8	7.2	7.6	8.6	12.7	11.1	9.9	10.3	8.0	5.9	100
Fort Stanton, N. Mex.	" " – " "	" "	5.1	5.1	6.7	9.5	12.3	14.1	12.2	15.1	5.1	5.2	4.7	4.9	100
Santa Fé, N. Mex.	" " – " "	" "	3.7	4.3	5.3	8.5	11.0	16.2	11.5	12.3	8.3	8.4	7.1	3.4	100
Average			4.5	5.5	7.0	8.2	10.0	11.6	12.4	11.6	9.0	8.3	7.0	4.9	100
Maximum departures			+1.4	+1.5	+1.5	+2.0	+3.1	+5.0	+5.1	+2.2	+2.4	+2.3	+2.1	+1.3	
±			−1.2	−2.4	−1.7	−1.5	−2.4	−3.0	−1.6	−2.2	−3.9	−3.1	−2.3	−1.8	
RECORDS BY MEASUREMENTS IN PANS.															
Austin, Tex.	Year of 1911	Floating-pan, 30 in.	4.9	3.4	5.2	6.0	9.2	15.6	14.4	16.9	13.1	5.6	3.0	2.7	100
El Paso, Tex. (Ft. Bliss).	May, 1889-Apr., 1890.	" " 3 ft.	2.4	2.4	8.3	8.7	12.7	12.7	11.4	13.5	10.9	8.1	5.5	3.4	100
	June, 1889-May, 1890.	" " "	2.4	2.4	8.3	8.7	12.7	12.7	11.4	13.5	10.9	8.1	5.5	3.4	100
	July, 1889-June, 1890.	" " "	2.3	2.3	8.2	8.6	12.7	13.6	11.3	13.4	10.8	8.0	5.4	3.4	100
	Aug., 1889-July, 1890.	" " "	2.3	2.3	8.2	8.6	12.7	13.6	11.3	13.4	10.8	8.0	5.4	3.4	100
	Sept., 1889-Aug., 1890	" " "	2.5	2.5	8.6	9.0	13.2	14.4	11.7	9.3	11.3	8.3	5.6	3.6	100
Albuquerque, N. Mex.	Feb., 1900-Jan., 1901.	Land-pan, 2 ft.	2.6	3.4	7.9	8.8	12.8	16.3	15.2	13.1	10.3	5.6	2.2	1.8	100
	Year of 1903.	" " "	2.1	2.4	5.9	11.4	12.5	12.9	14.1	13.3	11.0	7.5	4.8	2.1	100
Elephant Butte, N. Mex.	July, 1909-June, 1910	Land-pan, 4 ft.	2.9	3.2	5.2	9.2	13.2	15.5	13.3	12.0	9.9	7.8	4.4	3.4	100
Lake Avalon, N. Mex.	July, 1909-June, 1910	Floating-pan, 4 ft.	4.8	4.8	5.8	7.9	10.7	11.2	13.6	12.8	10.1	7.9	6.1	5.3	100
Average			2.9	2.9	7.2	8.7	12.2	13.8	12.8	13.1	10.9	7.5	4.8	3.2	100
Maximum departures			+2.0	+1.9	+1.4	+2.7	+1.0	+2.5	+2.4	+3.8	+2.2	+0.8	+1.3	+2.1	
±			−0.8	−0.6	−2.0	−2.7	−3.0	−2.6	−1.5	−3.8	−1.0	−1.9	−2.6	−1.4	

For data from which above percentages were calculated see Table 17.

TABLE 21.—Cumulative Evaporation and Temperature at Stations for Which Evaporation was Measured in Texas and New M

Station.	El Paso, Tex. (at Ft. Bliss).				El Paso, Tex. (at Ft. Bliss).				Elephant Butte, N. Mex.				Lake Avalon (Carlsbad, N. Mex.).				Carlsbad, N. Mex.				Carlsbad, N. Mex.				Albuquerque, N.		
Elevation	3 700 ft.±				3 700 ft.±				4 250 ft.±				3 200 ft.±				3 000 ft.±				3 000 ft.±				5 000 ft.±		
Character	Floating-pan, 3 ft.				Floating-pan, 3 ft.				Land-pan, 4 ft., modified to 3-ft., floating.*				Floating-pan, 4 ft., modified to 3 ft.†				Land-pan, 3 ft., modified to floating.‡				Land-pan, 3 ft., modified to floating.§				Land-pan, 2 ft., mod floating, 3 ft.		
Period observed	June, 1889–May, 1890, incl.				Sept., 1889–Aug., 1890, incl.				July, 1909–June, 1910, incl.				July, 1909–June, 1910, incl.				Jan.–Dec., 1899.				Jan.–Dec., 1901.				Feb., 1900–Jan.,		
Months.	Evaporation.		Temp. (F°).		Evaporation.		Temp. (F°).		Evaporation.		Temp. (F°).		Evaporation.		Temp. (F°).		Evaporation.		Temp. (F°).		Evaporation.		Temp. (F°).		Evaporation.		Tem
	Inches.	Cumulative.	Monthly mean.	Cumulative degree-days.	Inches.	Cumulative.	Monthly mean.	Cumulative degree-days.	Inches.	Cumulative.	Monthly mean.	Cumulative degree-days.	Inches.	Cumulative.	Monthly mean.	Cumulative degree-days.	Inches.	Cumulative.	Monthly mean.	Cumulative degree-days.	Inches.	Cumulative.	Monthly mean.	Cumulative degree-days.	Inches.	Cumulative.	Monthly mean.
January	2.0	2.0	48.3	1 497	2.0	2.0	48.3	1 497	2.3	2.3	42.5	1 316	4.9	4.9	46.4	1 438	1.01	1.01	42.3	1 308	0.83	0.83	44.7	1 386	1.51	1.51	35.4
February	2.0	4.0	51.8	2 948	2.0	4.0	51.8	2 948	2.6	4.9	44.2	2 555	4.9	9.8	47.6	2 771	1.01	2.02	42.8	2 507	0.83	1.66	47.0	2 702	1.95	3.46	40.6
March	7.0	11.0	58.8	4 771	7.0	11.0	58.8	4 771	4.2	9.1	57.2	4 328	5.9	15.7	61.3	4 672	2.51	4.53	58.4	4 317	2.06	3.72	56.9	4 466	4.56	8.02	49.1
April	7.3	18.3	64.3	6 699	7.3	18.3	64.3	6 699	7.4	16.5	61.6	6 176	8.0	23.7	64.4	6 608	3.08	7.56	63.8	6 231	1.40	5.12	61.1	6 299	5.05	13.07	52.0
May	10.8	29.1	75.2	9 031	10.8	29.1	75.2	9 031	10.7	27.2	71.2	8 384	10.9	34.6	72.3	8 845	4.46	12.02	72.6	8 482	4.00	9.12	71.9	8 528	7.46	20.53	67.0
June	10.7	39.8	80.8	11 454	11.7	40.8	79.6	11 419	12.5	39.7	78.8	10 747	11.9	46.5	81.2	11 281	7.21	19.23	78.2	10 828	5.40	14.52	81.4	10 909	9.35	29.88	76.3
July	9.6	49.4	83.0	14 028	9.6	50.4	82.3	13 970	10.8	50.5	81.1	13 362	13.9	60.4	82.0	13 823	6.42	25.65	80.6	13 326	5.20	19.72	80.2	13 456	8.72	38.60	79.1
August	11.4	60.8	82.7	16 591	7.6	58.0	78.8	16 413	9.8	60.3	79.2	15 717	13.0	73.4	79.0	16 272	6.32	31.97	83.6	15 918	5.80	25.52	81.6	15 985	7.56	46.16	75.0
September	9.2	70.0	71.4	18 733	9.2	67.2	71.4	18 555	8.1	68.4	70.7	17 838	10.2	83.6	71.8	18 426	4.48	36.45	74.5	18 153	3.80	29.32	74.1	18 208	5.92	52.08	67.7
October	6.8	76.8	65.0	20 748	6.8	74.0	65.0	20 570	6.3	74.7	61.0	19 729	7.5	91.1	63.2	20 385	3.36	39.81	63.6	20 124	3.00	32.32	66.6	20 273	3.24	55.32	58.8
November	4.6	81.4	49.0	22 218	4.6	78.6	49.0	22 040	3.6	78.3	51.8	21 363	6.2	97.3	57.2	22 101	2.57	42.38	53.4	21 726	1.38	33.70	56.6	21 971	1.28	56.60	44.7
December	2.9	84.3	53.2	23 368	2.9	81.5	53.7	23 689	2.8	81.1	34.4	22 349	4.9	102.2	39.6	23 329	1.11	43.49	43.2	23 066	0.91	34.61	46.8	23 422	1.04	57.64	34.6
Total and average	84.3	84.3	65.3	23 368	81.5	81.5	64.8	23 689	81.1	81.1	61.1	23 349	102.2	102.2	63.8	23 329	43.49	43.49	63.1	23 066	34.61	34.61	64.1	23 422	57.64	57.64	56.5

* Recorded Evap. × 1.08 × 0.80. † Recorded Evap. × 1.06. ‡ Land-pan Record × 80%. § Land-pan Record × 80%. ‖ Land-pan Record × 0.

The data in Table 21 are taken from Table 17 (Plate LXII), in which full notes and references are given.

1.—Cumulative Evaporation and Temperature at Stations for Which Evaporation was Measu

Bliss).	Elephant Butte, N. Mex.				Lake Avalon (Carlsbad, N. Mex.).				Carlsbad, N. Mex.				Carlsbad, N. M			
	4 250 ft.±				3 200 ft.±				3 000 ft.±				3 000 ft.±			
.	Land-pan, 4 ft., modified to 3-ft., floating.*				Floating-pan, 4 ft., modified to 3 ft.†				Land-pan, 3 ft., modified to floating.‡				Land-pan, 3 ft., mo floating.§			
incl.	July, 1909–June, 1910, incl.				July, 1909–June, 1910, incl.				Jan.–Dec., 1899.				Jan.-Dec., 19			
(F°).	Evaporation.		Temp. (F°).		Evaporation.		Temp. (F°).		Evaporation.		Temp. (F°).		Evaporation.		Te	
Cumulative degree-days.	Inches.	Cumulative.	Monthly mean.	Cumulative degree-days.	Inches.	Cumulative.	Monthly mean.	Cumulative degree-days.	Inches.	Cumulative.	Monthly mean.	Cumulative degree-days.	Inches.	Cumulative.	Monthly mean.	
1 497	2.3	2.3	42.5	1 316	4.9	4.9	46.4	1 438	1.01	1.01	42.8	1 308	0.83	0.83	44.	
2 948	2.6	4.9	44.2	2 555	4.9	9.8	47.6	2 771	1.01	2.02	42.8	2 507	0.83	1.66	47.	
4 771	4.2	9.1	57.2	4 328	5.9	15.7	61.3	4 672	2.51	4.53	58.4	4 317	2.06	3.72	56.	
6 699	7.4	16.5	61.6	6 176	8.0	23.7	64.4	6 608	3.03	7.56	63.8	6 231	1.40	5.12	61.	
9 081	10.7	27.2	71.2	8 384	10.9	34.6	72.3	8 845	4.46	12.02	72.6	8 482	4.00	9.12	71.	
11 419	12.5	39.7	78.8	10 747	11.9	46.5	81.2	11 281	7.21	19.23	78.2	10 828	5.40	14.52	81.	
13 970	10.8	50.5	81.1	13 262	13.9	60.4	83.0	13 823	6.42	25.65	80.6	13 326	5.20	19.72	80.	
16 413	9.8	60.3	79.2	15 717	13.0	73.4	79.0	16 272	6.32	31.97	83.6	15 918	5.80	25.52	81.	
18 555	8.1	68.4	70.7	17 838	10.2	83.6	71.8	18 426	4.48	36.45	74.5	18 153	3.80	29.32	74.	
20 570	6.3	74.7	61.0	19 729	7.5	91.1	63.2	20 385	3.36	39.81	63.6	20 124	3.00	32.32	66.	
22 040	3.6	78.3	51.8	21 283	6.2	97.3	57.2	22 101	2.57	42.38	53.4	21 726	1.38	33.70	56.	
23 689	2.8	81.1	34.4	22 349	4.9	102.2	39.6	23 329	1.11	43.49	43.2	23 066	0.91	34.61	46.	
23 689	81.1	81.1	61.1	22 349	102.2	102.2	63.8	23 329	43.49	43.49	63.1	23 066	34.61	34.61	64.	

* Recorded Evap. × 1.08 × 0.80. † Recorded Evap. × 1.08. ‡ Land-pan Record × 80%. § Land-pan Record

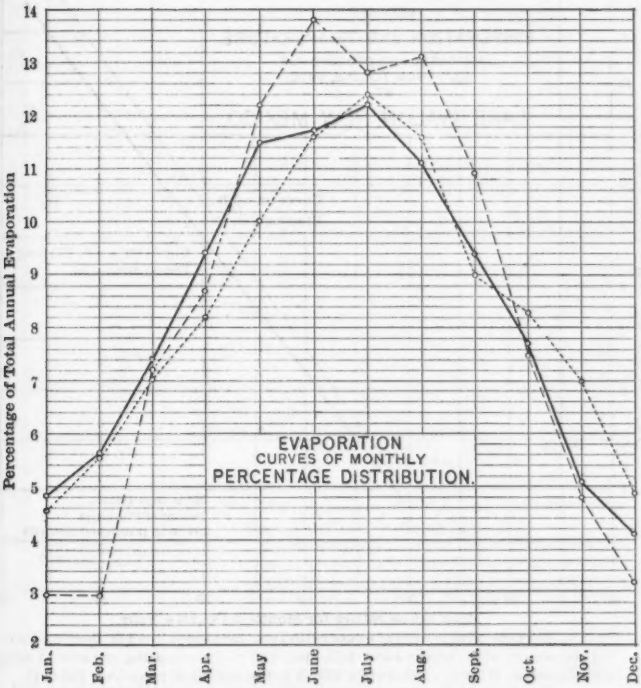
The data in Table 21 are taken from Table 17 (Plate LXII), in which full notes and references are given.

EASURED IN TEXAS AND NEW MEXICO.

D, N. MEX.		ALBUQUERQUE, N. MEX.				ALBUQUERQUE, N. MEX.							
0 ft.±		5 000 ft.±				5 000 ft.±							
ft., modified to ting.§		Land-pan, 2 ft., modified to floating, 3 ft.‖				Land-pan, 2 ft., modified to floating-pan, 3 ft. ¶							
ec., 1901.		Feb., 1900–Jan., 1901.				January 1, 1903–July 1, 1904.							
Temp. (F°).		Evaporation.		Temp. (F°).		Evaporation.		Temp. (F°).		Evaporation.		Temp. (F°).	
Monthly mean.	Cumulative degree-days.	Inches.	Cumulative.	Monthly mean.	Cumulative degree-days.	Inches.	Cumulative.	Monthly mean.	Cumulative degree-days.	Inches.	Cumulative.	Monthly mean.	Cumulative degree-days.
44.7	1 386	1.51	1.51	35.4	1 097	1.34	1.34	36.3	1 125	1.69	66.73	36.4	20 905
47.0	2 702	1.95	3.46	40.6	2 234	1.53	2.87	34.6	2 094	2.90	69.63	50.8	22 378
56.9	4 466	4.56	8.02	49.1	3 756	3.86	6.73	47.3	3 557	5.54	75.17	57.2	24 151
61.1	6 299	5.05	13.07	52.0	5 316	7.49	14.16	56.5	5 252	7.93	83.10	58.2	25 897
71.9	8 528	7.46	20.53	67.0	7 393	8.12	22.28	69.4	7 186	9.27	92.37	65.0	27 912
81.4	10 969	9.35	29.88	76.3	9 682	8.38	30.66	68.4	9 238	9.64	102.01	71.3	30 051
80.2	13 458	8.72	38.60	79.1	12 134	9.15	39.81	75.2	11 509				
81.6	15 985	7.56	46.16	73.0	14 459	8.68	48.49	74.2	13 869				
74.1	18 208	5.92	52.08	67.7	16 490	7.14	55.63	64.0	15 789				
66.6	20 275	3.24	55.32	56.8	18 251	4.90	60.53	53.9	17 460				
56.6	21 971	1.28	56.60	44.7	19 592	3.12	63.65	42.4	18 732				
46.8	23 422	1.04	57.64	34.6	20 665	1.39	65.04	33.7	19 777				
64.1	23 422	57.64	57.64	56.5	20 665	65.04	65.04	54.1	19 777				

Record × 80%. ‖ Land-pan Record × 0.80 × $\frac{1}{1.08}$. ¶ Land-pan Record × 0.80 × $\frac{1}{1.08}$.

However, even with the reduction of the year's evaporation at Lake Avalon to, say, 86 to 87 in., its evaporation still would be so much greater than that of Carlsbad as to leave some doubt as to the true evaporation in this vicinity. The most probable explanation of the disagreement between the Lake Avalon and the Carlsbad records seems to be that during the particular year under discussion the evaporations



---------Mean distribution from records of eleven stations in Texas and New Mexico taken with Piche evaporimeter in 1887-88.

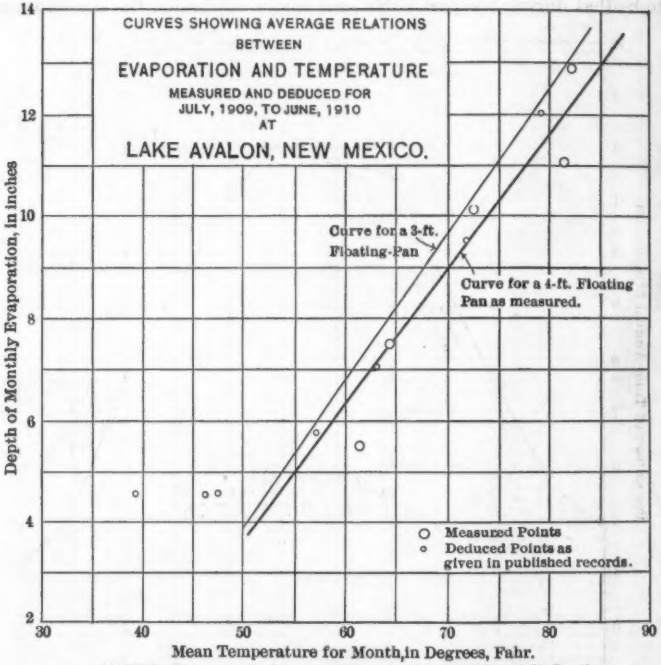
— — — —Mean distribution from records of measured evaporation at five stations in Texas and New Mexico for various years.

——— Distribution at La Boquilla, April, 1912, December, 1913, as calculated by temperture-evaporation relations.

FIG. 11.

at the two places (not far apart) were abnormal, those at Lake Avalon being abnormally high and those at Carlsbad abnormally low. In considering hereinafter the effect of elevation on temperature, the means of the Carlsbad and Lake Avalon evaporations are used as the Carlsbad values, and all other considerations of the Lake Avalon records are omitted.

The curves showing the relations between the mean monthly temperatures and the monthly evaporation depths at Austin, El Paso, Albuquerque, Carlsbad, and Elephant Butte, shown separately on Figs. 6 to 10, also are shown together for comparison on Diagram I of Fig. 13.



NOTES: This record of evaporation was measured in a 4-ft. floating-pan for the season, July, 1909, to June, 1910, incl. The pan was floating in Lake Avalon, near Carlsbad, N. Mex. at Elevation 3200 ft. ±. For details of record see Table 17.

A portion of the season's record as given was deduced, but the published record does not state the manner in which this was done. The total evaporation during the months of actual measurement was for (4-ft. pan)_____ 47.01 in. and for the same months by the average curve shown above (for 4-ft. pan) 47.7 in.

FIG. 12.

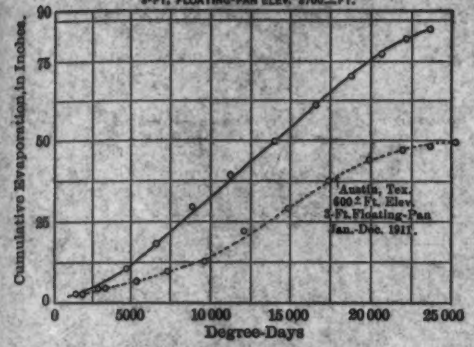
Piche Evaporimeter Records.—In addition to the six stations for which pan evaporations are given in Table 17 (Plate LXII), that table presents evaporimeter observations at one of the six (El Paso) and at ten other stations in Texas and New Mexico. These evaporimeter records* are stated to have been published in the *Monthly Weather Review* of the U. S. Signal Service, September, 1888, p. 235,

* "Public Water Supplies", Turneaure and Russell (1911), Second Edition, p. 60.

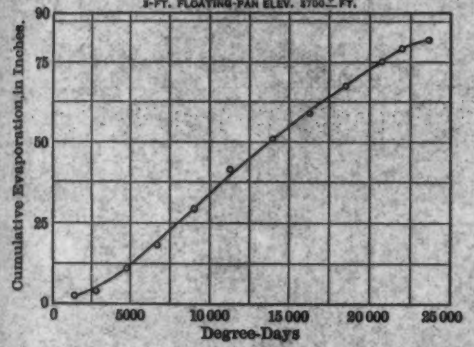
PLATE LXV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXX, No. 1376.
DURYEA AND HAEHL ON
EVAPORATION FROM LAKE CONCHOS, MEXICO.

DIAGRAMS SHOWING
RELATIONS BETWEEN
EVAPORATION
AND
TEMPERATURE
CONSIDERED CUMULATIVELY
AT STATIONS WHERE MEASURED
IN TEXAS AND NEW MEXICO.

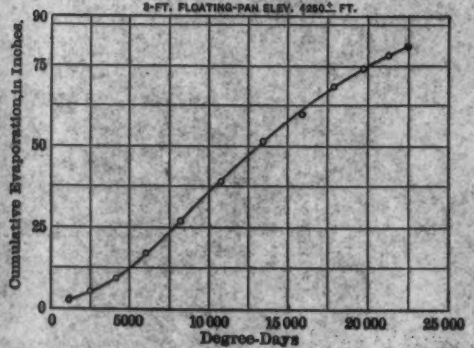
EL PASO, TEX. JUNE 1889-MAY 1890
3-FT. FLOATING-PAN ELEV. 3700 ± FT.



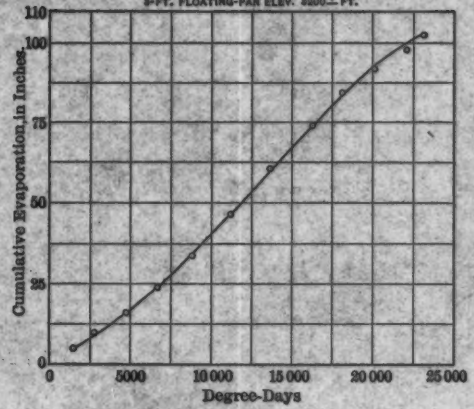
EL PASO, TEX. SEPT. 1889-AUG. 1890
3-FT. FLOATING-PAN ELEV. 3700 ± FT.



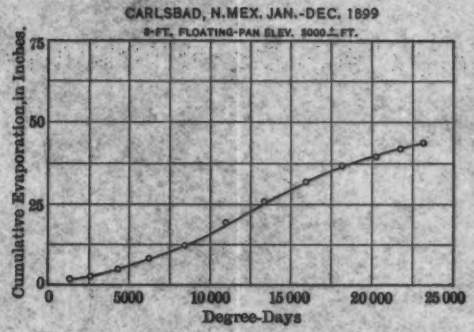
ELEPHANT BUTTE, N.MEX. JULY, '09-JUNE, '10
3-FT. FLOATING-PAN ELEV. 4250 ± FT.



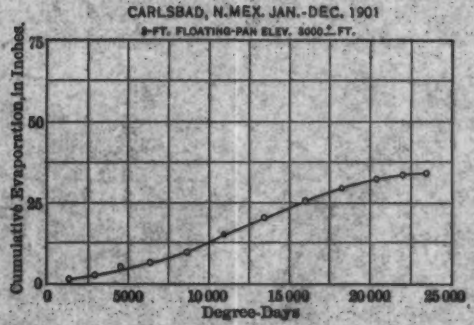
LK. AVALON, N.MEX. JULY, '09-JUNE, '10
3-FT. FLOATING-PAN ELEV. 3200 ± FT.



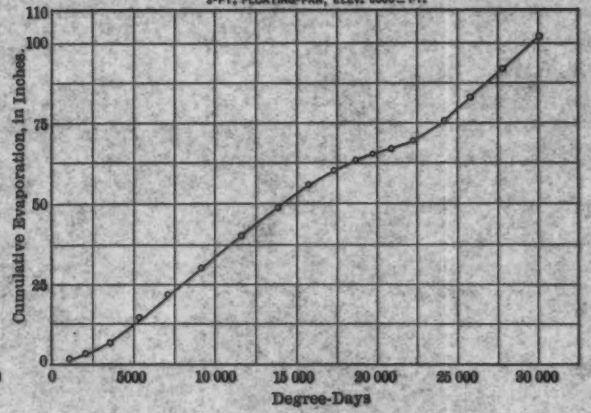
CARLSBAD, N.MEX. JAN.-DEC. 1899
3-FT. FLOATING-PAN ELEV. 3000 ± FT.



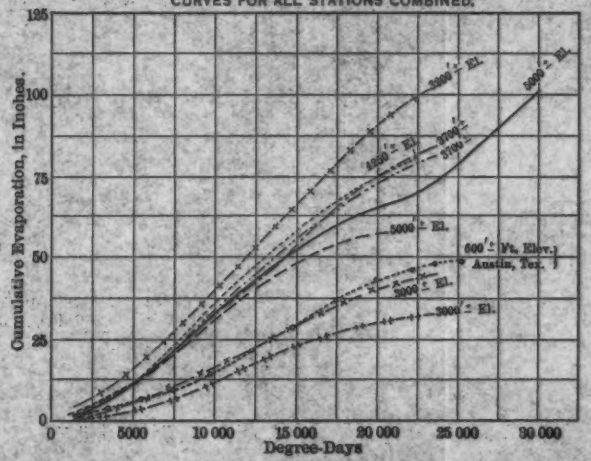
CARLSBAD, N.MEX. JAN.-DEC. 1901
3-FT. FLOATING-PAN ELEV. 3000 ± FT.



ALBUQUERQUE, N.MEX. JAN. 1903-JULY, 1904
3-FT. FLOATING-PAN, ELEV. 5000 ± FT.



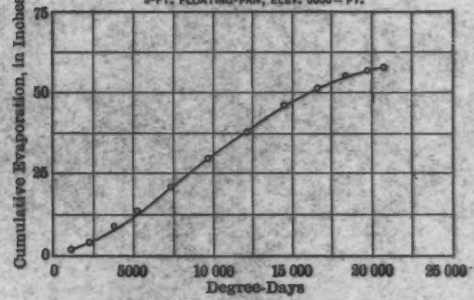
EVAPORATION-TEMPERATURE RELATIONS
CURVES FOR ALL STATIONS COMBINED.



DESCRIPTION

- Lake Avalon, N.Mex., July 1909-June, '10.
- Elephant Butte, July 1909-June 1910.
- El Paso, Tex., June 1889-May 1890.
- El Paso, Tex., Sept. 1889-Aug. 1890.
- Albuquerque, New Mex., Jan. 1903-July, 1904.
- Albuquerque, New Mex., Feb. 1900-Jan. 1901.
- Carlsbad N.Mex., January-Dec. 1899.
- Carlsbad N.Mex., January-Dec. 1901.

ALBUQUERQUE, N.MEX. FEB. 1900-JAN. 1901
3-FT. FLOATING-PAN, ELEV. 5000 ± FT.



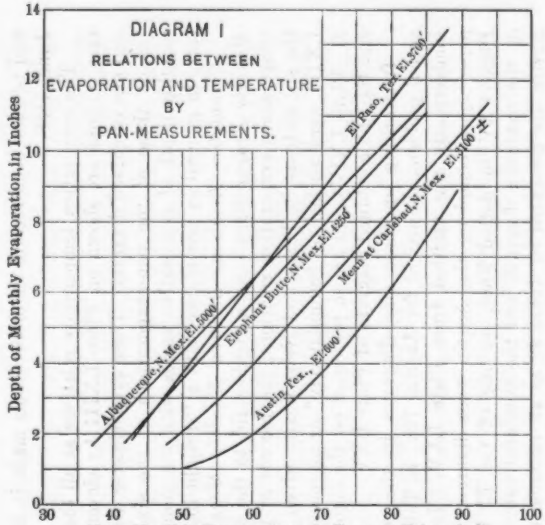
NOTES.

The diagrams on this Plate are plotted from the data of Table 21 (Plate LXIV).

The measured depths of evaporation as given in that Table have been modified by the application of coefficients, in order that all records may conform to depths for a 3-ft. floating-pan, and thus be directly comparable.

All records are plotted hereon beginning with January and ending with December, of the year observed, except that of Albuquerque, N. Mex., for 1903-04 which extends to July, 1904.

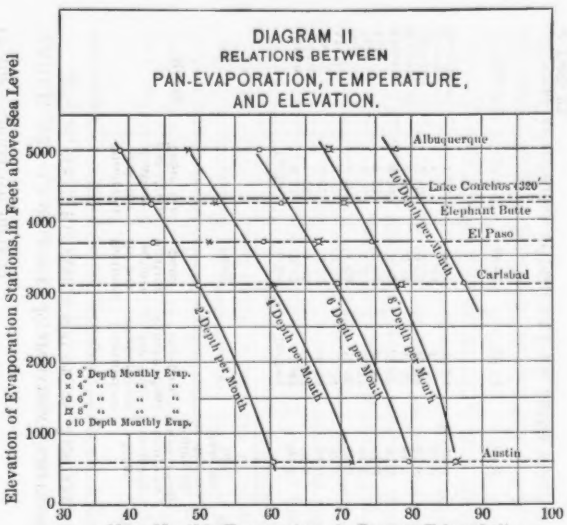
Committee Packages Delivered



NOTE:- On Diagram I are plotted the curves of relation between Evaporation and Temperature for all stations in Southern Texas and New Mexico at which pan-evaporation measurements are known to have been made.

For reasons stated in the text, the record for Carlsbad, N. Mex., shown hereon is the mean of the curves resulting from records at Carlsbad in 1899 and 1901, and at Lake Avalon, near Carlsbad, in 1909-10.

Diagram I shows that, in general, the rate of evaporation at any temperature increases with elevation.



NOTE:- Diagram II is derived from Diagram I, and shows that the mean monthly temperature required to produce a given depth of monthly evaporation decreases with elevation.

The curves show a definite relation between the elevation and temperature necessary to produce any given evaporation depth, to which all stations conform closely except El Paso, Tex. The record at that station, though covering a period of 34 months, is not complete for any month, the missing days of each month having been deduced in every case. Although portions of the records at most other stations were also deduced, it is believed to be proper to give greatest weight to those records which conform to definite relations, and the curves of Diagram II have been so drawn.

FIG. 13.

and to have been records of observations made in the year, July, 1887-June, 1888.

The evaporation temperature relations at all these eleven evaporimeter stations are shown on Plate LXIII by eleven separate diagrams and one combined diagram. These evaporimeter diagrams show clearly (as do those of the pan-evaporations at the six stations) that the monthly depth of evaporation varies with the monthly mean temperature, in general in accordance with a "straight-line" relationship.

Cumulative Relations Between Monthly Mean Temperatures and Monthly Evaporations.—As a further aid to the study of the law of temperature evaporation relations, the records of pan evaporations at the six stations in Texas and New Mexico given in Table 17 (Plate LXII) were worked up "cumulatively" or as mass-diagrams, in Table 21 (Plate LXIV) and are so presented on the eight separate diagrams and the one combined diagram of Plate LXV.

In working up these mass-diagrams in the consideration of the actual problem, the record for Austin, Tex., in Table 17 (Plate LXII) inadvertently was omitted from Plate LXV. To make the present paper more complete, the Austin mass-diagram has been added to Plate LXV in dotted lines, on one of the El Paso diagrams and on the combined diagram, and the tabulated cumulative relations for Austin are presented in Table 22 as an example of the working out of the cumulative method.

TABLE 22.—CUMULATIVE EVAPORATION-TEMPERATURE RELATIONS AT AUSTIN, TEX., ELEVATION 600 FT. ±, JANUARY-DECEMBER, 1911.

Month.	Observed in 30-in. floating-pan (from Table 17).	Deduced for 3-ft. square floating-pan.	Cumulative evaporation for 3-ft square floating-pan.	Mean monthly temperature (from Table 17), in degrees, Fahrenheit.	Degree-Days: (56.5° × 31, 58.8° × 28, 64.0° × 31, etc.).	
	104%	100%	(*A*)*		For month	Cumulative (*B*)*
	Inches.	Inches.	Inches.			
January	2.52	2.42	2.42	56.5	1 750	1 750
February	1.72	1.65	4.07	58.8	1 647	3 397
March	2.62	2.52	6.59	64.0	1 983	5 380
April	3.07	2.95	9.54	67.0	2 010	7 390
May	4.70	4.52	14.06	73.3	2 272	9 662
June	7.96	7.65	21.71	84.2	2 524	12 186
July	7.33	7.04	28.75	84.2	2 609	14 795
August	8.58	8.25	37.00	86.3	2 674	17 469
September	6.69	6.43	43.43	84.2	2 524	19 993
October	2.86	2.75	46.18	67.8	2 102	22 095
November	1.51	1.45	47.63	55.0	1 650	23 745
December	1.36	1.31	48.94	49.0	1 519	25 264
Year	50.92	48.94	48.94	69.2	25 264	25 264

* The Austin curves of Plate LXV are plotted from the values, (*A*) and (*B*), of Table 22.

It is noticeable that all the curves of Plate LXV (which show the relations between the cumulative evaporation depths in inches and the corresponding "degree-days") are of a similar "ogee" form at all the measured stations, and for a whole year do not depart greatly from a straight line. The departures from the straight line, showing greater rates of evaporation per degree-day for some months than for others, probably are due to other conditions than temperature which affect evaporation, such as the more frequent, more sustained, and more severe winds of some months, the lower humidity of the air during some months, etc., etc.

From Figs. 6 to 10, Fig. 12, and Plates LXIII and LXIV, and the discussions relating to them, the conclusion seems warranted that (at least for the Great Plateau and for the complete cycle, averaging a year) the monthly depth of evaporation increases with the mean monthly temperature, usually by an approximate "straight-line" relation.

Subsidiary Conclusion (*e*):

Relations Between Evaporation Depth and Elevation Above Sea.

The relations between elevation and evaporation depth are shown and studied on the combined diagrams of Plates LXIII and LXIV and on Fig. 13; but most plainly by Diagram II of Fig. 13, and by Fig. 4 (which was not used in the investigation, but was made only for this paper).

Since, at elevations above sea level, water boils at temperatures below 212° Fahr., it appears at least reasonable that at any given temperature water will vaporize or evaporate more quickly at high than at low elevations.

On Diagram I of Fig. 13 are plotted the curves showing the relations between temperature and evaporation in 3-ft. square floating-pans at five measured evaporation stations on the Great Plateau. On each curve is marked the name and elevation of its station; and this diagram shows in a general way that at equal mean monthly temperatures the evaporation depth is greater at stations of higher elevation. The combined diagram of the Piche evaporimeter observations (Plate LXIII) also shows this in a general way.

However, the increases of evaporation depth with increases both of mean temperature and of elevation are best shown by Diagram II of Fig. 13 (made up from Diagram I), by Tables 23, 24, and 25 (made up by reading intersection values from Diagram II, Fig. 13), and by Fig. 4 (plotted from the values of Tables 23 and 25).

In the study made for Fig. 4, the values of Table 24 were extended upward to Elevation 7 000 ft., downward to 500 ft. below sea level (say as for Salton Sea and Dead Sea), and to the 2-in. increment of 16 to 18 in. monthly evaporation depth; and the values of Table 23

TABLE 23.—MEAN MONTHLY TEMPERATURES CAUSING STATED DEPTHS OF MONTHLY EVAPORATION FROM 3-FT. SQUARE FLOATING-PANS, AT VARIOUS ELEVATIONS ABOVE SEA LEVEL: FOR STATIONS ON GREAT PLATEAU ONLY.

(Values are read from curves of Diagram II, Fig. 13.)

Elevation above sea level, in feet.	MONTHLY DEPTHS OF EVAPORATION.					
	2 in.	4 in.	6 in.	8 in.	10 in.	12 in.
	Monthly Mean Temperature, in Degrees, Fahrenheit.					
7 000	*(26.5°)	(34.7°)	(44.9°)	(55.6°)	(67.3°)	(79.7°)
6 000	(32.7)	(41.7)	(51.5)	(61.6)	(72.0)	(82.9)
5 000	38.8	48.4	57.9	67.5	76.9	(86.3)
4 000	44.7	54.8	64.0	73.1	†82.8 (81.3°)	(89.1)
3 000	50.3	60.7	69.6	78.1	†88.0 (85.2°)	(91.4)
2 000	55.2	66.0	74.6	82.4	(88.4)	(93.2)
1 000	59.1	70.2	78.6	85.4	(90.4)	(93.6)
600	60.4	71.6	79.8	86.3	(90.8)	(93.5)
000	(61.8)	(73.1)	(81.1)	(87.3)	(91.2)	(93.0)

* No actual evaporation data were available above Elevation 5 000 ft., below Elevation 600 ft., or for monthly evaporation depths greater than 10 in.; and all values in Table 23 beyond those limits (shown in parentheses, as (26.5°), etc.) were filled in by use of the supposititious increments of Table 24.

† From the study embodied in Table 24, the observed values in Table 23 for elevations of 4 000 and 3 000 ft. and for 10 in. evaporation (†82.8° and †88.0°) appeared to be abnormal, and hence were replaced (in Tables 23, 24, and 25, and in Fig. 4) by the adjusted supposititious values (81.3°) and (85.2°).

TABLE 24.—INCREMENTS IN MEAN MONTHLY TEMPERATURES CORRESPONDING TO 2-IN. INCREMENTS IN DEPTH OF MONTHLY EVAPORATION FROM 3-FT. SQUARE FLOATING-PANS, ON THE GREAT PLATEAU.

Elevation above sea level, in feet.	2-IN. INCREMENTS IN MONTHLY EVAPORATIONS.				
	2 to 4 in.	4 to 6 in.	6 to 8 in.	8 to 10 in.	10 to 12 in.
	Corresponding Increments in Monthly Mean Temperatures, in Degrees, Fahrenheit.				
7 000	*(8.2°)	(10.2°)	(10.7°)	(11.7°)	(12.4°)
6 000	(9.0)	(9.8)	(10.1)	(10.4)	(10.9)
5 000	9.6	9.5	9.6	9.4	(9.4)
4 000	10.1	9.2	9.1	†9.7 (8.2°)	(7.8)
3 000	10.4	8.9	8.5	†9.9 (7.1°)	(6.2)
2 000	10.8	8.6	7.8	(6.0)	(4.8)
1 000	11.1	8.4	6.8	(5.0)	(3.2)
600	11.2	8.2	6.5	(4.5)	(2.7)
000	(11.3)	(8.0)	(6.2)	(3.9)	(1.8)

* All values in this table not in parentheses are first differences from the corresponding values of Table 23. All values in parentheses, as *(8.2°), etc., are deduced by a more or less careful extension or projection of the first differences not in parentheses, by the aid of second and third differences, and studies of the same by diagrams.

† As explained in Table 23, the two values, †9.7° and †9.9°, are believed to be abnormal, and hence are replaced in use by the adjusted values (8.2°) and (7.1°).

were extended to the same limits. This was done because it was surmised that the combined influence on evaporation depth of all other conditions except temperature and elevation might be relatively unimportant—perhaps so small as to permit the estimation of approximate evaporation losses directly from Fig. 4, not only for places on the Great Plateau, but even generally throughout the United States.

However, the diagram of the extended Table 23 showed serious contradictions beyond 12 in. monthly evaporation (*e. g.*, at 400 ft. below sea level and 81.4° mean monthly temperature, the 6 and 18-in. lines intersect and give either 6 or 18 in. monthly evaporation), and hence it shows that values derived by extensions so far beyond the observed limits are very unreliable.

Also, comparisons were made of the evaporation losses shown by such extended Fig. 4, with the observed evaporations at Salton Sea (263 ft. below sea level), Lake Tahoe (6 225 ft. elevation), and Coyote or Laguna Seca (240 ft. elevation)—all in California and outside the limits of the Great Plateau—and all such comparisons showed very serious disagreements, and indicated that Tables 23, 24, and 25, and Fig. 4 are not applicable with safety to places outside the limits of the Great Plateau.

In drawing up Fig. 4 from Tables 23 and 25, the diagrams were restricted nearly to the limits of the observed evaporations (from 600 to 5 000 ft. elevation and from 2 to 10 in. monthly evaporation), though it is believed that, for stations on the Great Plateau, the diagrams may be extended with reasonable accuracy to elevations of 7 000 ft., and to monthly evaporation depths (in 3-ft. square floating-pans) of 12 in. In Tables 23, 24, and 25, the values are extended downward to Elevation 000 ft. or sea level, necessarily outside the limits of the Plateau.

Table 23 is plotted as Diagram (*a*), and Table 25 as Diagram (*b*), of Fig. 4. Table 25 is derived by reading intersection values from Diagram II of Fig. 13 and from the (extended) Diagram (*a*) of Fig. 4. Most of the explanations given of Tables 23 and 24 apply also to Table 25. Some parts of Diagram (*b*) of Fig. 4 are not in close agreement with Diagram (*a*), because of slight graphical adjustment to straight-line relations.

In connection with the discussion of Fig. 4, reference should be made to the yearly evaporation depth shown at the upper edge of Diagram (*b*). The mean temperature for the year is the average of the twelve mean monthly temperatures, and the evaporation temperature relations of Diagram (*b*) are quite generally "straight-line" relations. Hence, if the temperature lines of that diagram are used as yearly mean temperatures and the yearly evaporation depth is read off at the top of the diagram (where the scale is one-twelfth that at the bottom), then approximately the same yearly evaporation will be obtained as if the temperature lines are used as monthly mean tem-

peratures, the monthly evaporation depths read off at the bottom of the diagram, and their sum for the 12 months taken.

TABLE 25.—ELEVATIONS CORRESPONDING TO STATED MEAN MONTHLY TEMPERATURES AND STATED MONTHLY DEPTHS OF EVAPORATION: FOR STATIONS ON GREAT PLATEAU ONLY.

Mean monthly temperatures, in degrees, Fahrenheit.	Monthly Depths of Evaporation from 3-ft. square Floating-Pans.					
	2 in.	4 in.	6 in.	8 in.	10 in.	12 in.
	Elevation, above Sea Level, in Feet.					
30	*(6 410)	....	....	....	....	....
35	(5 610)	(6 960)	....	....	....	....
40	4 800	(6 240)	....	....	....	....
45	3 940	(5 500)	(6 980)	....	....	....
50	3 040	4 740	(6 210)	....	....	....
55	2 080	3 970	(5 460)	(7 090)	....	....
60	710	3 120	4 660	(6 270)	....	....
65	....	2 200	3 820	(5 420)	(7 500)	....
70	....	1 060	2 920	4 570	(6 430)	....
75	....	....	1 910	3 640	(5 390)	(8 460)
80	....	....	(510)	2 560	† 4 500 (4 310)	(6 910)
85	....	....	....	1 160	† 3 600 (3 090)	(5 380)
90	....	....	....	....	(1 300)	(3 660)

For * and †, see explanations following Tables 23 and 24.

TABLE 26.—SMALL VARIATIONS IN YEARLY MEAN TEMPERATURES, IN DEGREES, FAHRENHEIT.

Station and period.	Mean Yearly Temperatures.		
	Average mean yearly, of all years.	Maximum mean yearly, of all years.	Minimum mean yearly, of all years.
* Salton, Cal.; 12 years (1889-1900).....	76.9° = 100.%	79.7° = + 3.5%	73.3° = — 4.6%
† San José, " ; 26 " (1874-1899).....	58.1 = 100.	61.7 = + 6.2	55.2 = — 4.9
‡ Truckee, " ; 30 " (1871-1900).....	43.9 = 100.	48 8 = + 11.1	37.3 = — 15.0
Average of the three stations.....	100.%	+ 6.9%	— 8.2%

* At Salton Sea, 263 ft. below sea level.
† 12 miles from Coyote or Laguna Seca, and 100 ft. above sea level.
‡ 12 miles from Lake Tahoe, and 5 818 ft. above sea level.

In closing the discussion of Fig. 4, attention should be called to the comparatively small differences at any station in the yearly mean temperatures of different years, the variations being much smaller than those of the rainfall and the stream flow. This comparative constancy of yearly mean temperatures adds greatly to the reliability of estimates of evaporation made by their aid, as by Fig. 4. The small variation in the yearly mean temperature of different years is

shown by the three instances in Table 26; Tables 26 and 27 having been worked up in connection with the test mentioned of the attempted general application of Fig. 4, from data given by Mr. Alex. G. McAdie.*

Even the monthly mean temperatures (though less constant for different years than the mean yearly) are quite constant in comparison with rainfall and stream flow. This is shown by Table 27.

TABLE 27.—COMPARATIVELY SMALL VARIATIONS IN MONTHLY MEAN TEMPERATURES, IN DEGREES, FAHRENHEIT.

Station.	Period, in years.	Month.	Mean Monthly Temperatures, in Degrees, Fahrenheit.		
			Average mean monthly, of all years.	Maximum mean monthly, of all years.	Minimum mean monthly, of all years.
Salton, Cal....	12	Jan.....	55.7° = 100.%	65.7° = + 8.0%	49.1° = −11.7%
		Feb.....	58.8 = 100.	67.9 = +15.6	49.6 = −15.6
		Mar.....	66.0 = 100.	74.0 = +12.2	57.8 = −12.3
		Apr.....	76.5 = 100.	82.1 = + 7.3	70.3 = − 8.0
		May.....	83.1 = 100.	94.0 = +13.0	73.3 = −11.6
		June.....	93.8 = 100.	100.6 = + 7.2	86.9 = − 7.4
		July.....	98.9 = 100.	107.0 = + 8.2	94.6 = − 4.3
		Aug.....	97.2 = 100.	107.4 = +10.6	89.3 = − 8.2
		Sept.....	91.0 = 100.	99.9 = +11.0	85.8 = − 5.6
		Oct.....	79.1 = 100.	85.3 = + 7.9	72.6 = − 8.2
		Nov.....	66.8 = 100.	70.8 = + 6.0	59.1 = −11.5
		Dec.....	56.1 = 100.	66.3 = +18.2	46.2 = −17.6
		Average.	76.9° = (100.%)	85.1° = (+10.4%)	69.6° = (−10.2%)
San José, Cal..	26	Jan.....	48.2° = 100.%	57.7° = +20.0%	40.4° = −16.0%
		Feb.....	50.7 = 100.	54.8 = + 8.0	45.3 = −10.6
		Mar.....	53.6 = 100.	57.8 = + 7.8	48.5 = − 9.4
		Apr.....	56.4 = 100.	62.1 = +10.2	51.9 = − 8.0
		May.....	60.1 = 100.	67.7 = +12.8	50.1 = −16.5
		June.....	65.5 = 100.	76.1 = +16.1	60.5 = − 7.6
		July.....	66.7 = 100.	71.1 = + 6.9	64.7 = − 3.0
		Aug.....	66.4 = 100.	70.1 = + 5.8	63.3 = − 4.7
		Sept.....	64.7 = 100.	71.0 = + 9.8	62.0 = − 4.0
		Oct.....	60.2 = 100.	65.5 = + 9.0	56.3 = − 6.5
		Nov.....	54.1 = 100.	57.6 = + 6.6	48.5 = −10.2
		Dec.....	50.1 = 100.	57.9 = +15.7	46.1 = − 8.0
		Average.	58.1° = (100.%)	64.1° = (+10.7%)	53.1° = (− 8.7%)
Truckee, Cal...	30	Jan.....	25.3° = 100.%	32.9° = +30.0%	16.4° = −35.2%
		Feb.....	28.3 = 100.	39.0 = +37.7	21.4 = −24.5
		Mar.....	32.9 = 100.	42.0 = +27.9	25.7 = −21.8
		Apr.....	40.0 = 100.	50.3 = +25.8	25.7 = −35.7
		May.....	48.4 = 100.	57.7 = +19.3	37.6 = −22.3
		June.....	57.4 = 100.	70.5 = +23.0	48.2 = −16.0
		July.....	65.4 = 100.	73.1 = +12.0	53.0 = −19.0
		Aug.....	63.4 = 100.	73.2 = +15.6	52.5 = −17.0
		Sept.....	55.9 = 100.	61.6 = +31.0	50.5 = − 9.6
		Oct.....	45.1 = 100.	60.6 = +12.1	38.7 = −14.2
		Nov.....	36.5 = 100.	43.4 = +19.0	29.0 = −20.5
		Dec.....	28.7 = 100.	35.9 = +25.0	21.5 = −25.0
		Average.	43.9° = (100.%)	52.5° = (+23.2%)	35.0° = (−21.7%)

* In "Climatology of California", 1903, *Bulletin L* of the Weather Bureau, U. S. Dept. of Agriculture.

From Table 26, at Salton, San José, and Truckee, the maximum departure from the yearly mean temperature is only 15% and the average departure only about 8%; and from Table 27, the maximum departure from the monthly mean temperature is only about 38% and the average departure less than half that. In comparison, the extremes of rainfall and stream flow at any place usually are about two-fifths and two and one-half times the mean value.

Not only is the yearly mean temperature at any place reasonably constant, but the yearly evaporation depth also is quite constant from year to year, much more so than the rainfall or the stream flow. Without going into this question thoroughly, some data to confirm this view are given in Tables 28 and 29.

TABLE 28.—COMPARATIVELY CLOSE AGREEMENT OF YEARLY EVAPORATION DEPTHS IN SANTA CLARA VALLEY, CALIFORNIA, IN TWO DIFFERENT YEARS: MEASURED IN 3-FT. SQUARE PANS.*

Group and No. of Pan.	Year, 1904.		Year, 1905.			
	Year's evaporation depth.		Year's evaporation depth.		Departure from year, 1904.	
	Inches.	Percentage.	Inches.	Percentage.	—	+
Coyote Valley Group.						
Pan No. 1 (land-pan)	49.8	100	46.7	93.9	6.1%	.....
" " 2 "	49.7	100	45.8	92.3	7.7	.....
Laguna Seca Group.						
Pan No. 3 (land-pan)	53.8	100	55.4	103.0	....	3.0%
" " 4 "	62.3	100	60.4	97.0	3.0%	.....
" " 5 "	53.8	100	54.8	101.8	....	1.8
" " 6 (floating-pan)	39.4	100	43.1	109.4	....	9.4
" " 7 "	43.0	100	53.5	124.5	....	24.5
Upper Gorge Group.						
Pan No. 8 (land-pan)	41.7	100	52.3	125.5	....	25.5
" " 9 "	47.2	100	45.2	95.9	4.1	.....
" " 10 "	46.5	100	55.4	119.0	....	19.0
" " 11 (floating-pan)	34.3	100	30.4	88.7	11.3	.....
Averages	....	100	....	104.7	— 6.4%	+13.9%
Extremes	....	....	....	....	—11.3	+25.5

* Data from *Engineering News*, Vol. 67, pp. 380–383, February 29th, 1912. "California Evaporation Records", by Edwin Duryea, Jr., M. Am. Soc. C. E.

From Tables 28 and 29, respectively, the averages of the variations in the yearly evaporation depth are about 10 and 6%, and the maximum variations about 25 and 14 per cent.

Finally, quoting from Folwell*:

"Evaporation is seen to vary less than does rainfall, the greatest variation from the annual mean at Boston during sixteen years, for instance, being about 13%."

* "Water-Supply Engineering", 1906, p. 90.

TABLE 29.—SMALL VARIATIONS OF THE YEAR'S EVAPORATION DEPTH FROM YEAR TO YEAR, AS OBSERVED FOR SEVERAL SUCCESSIVE YEARS.

AT LAKE TAHOE, CALIFORNIA, AS MEASURED IN A 2-FT. SQUARE FLOATING-PAN.*					AT EMDRUP, DENMARK (NO DATA OF PAN).†				
Years (6).	Year's evaporation depth.		Departure from average mean yearly depth.		Years (11).	Year's evaporation depth.		Departure from average mean yearly depth.	
	Inches.	Percentage.	—	+		Inches.	Percentage.	—	+
1900-01	28.2	92.0	8.02%	....	1849	29.5	105.7	....	5.7%
1901-02	25.7	84.0	16.0	....	1850	29.1	104.2	....	4.2
1902-03	33.0	107.9	....	7.9%	1851	28.4	101.8	....	1.8
1903-04	31.3	102.2	....	2.2	1852	29.4	105.3	....	5.3
1904-05	33.9	110.8	....	10.8	1853	26.9	96.3	3.7%	....
1905-06	31.4	102.6	....	2.6	1854	27.9	100.0	0.0	0.0
					1855	25.1	89.9	10.1	....
					1856	24.0	86.0	14.0	....
					1857	29.9	107.2	....	7.2
					1858	30.6	108.1	....	8.1
					1859	26.4	94.5	5.5	....
Average...	30.6	100.0	−12.0%	+5.9%	Average...	27.9	100.0	−6.7%	+4.6%
Extremes..	....	.....	−16.0	+10.8	Extremes.	....	.....	−14.0	+8.1

* From *Engineering News*, Vol. 67, pp. 380–383.
† From Fanning's "Water-Supply Engineering", 1906, p. 89.

This closes the discussion and working out of the five subsidiary conclusions of this investigation. They are stated briefly and precisely in Table 2.

There remains the working out of the principal conclusion, the estimation of the yearly evaporation depth from Lake Conchos by five more or less independent methods.

PRINCIPAL CONCLUSION:

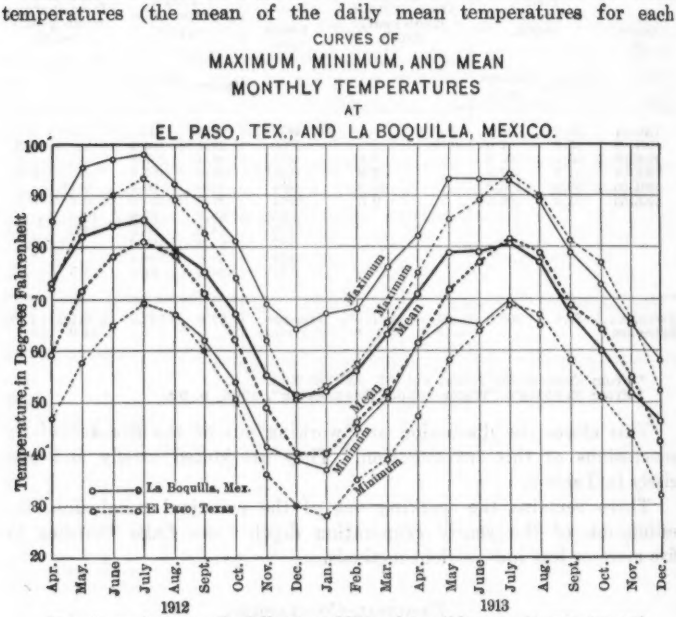
YEARLY DEPTH OF EVAPORATION FROM LAKE CONCHOS.

Method A.—This method consists of estimating the evaporation depth from Lake Conchos by means of Diagram II of Fig. 13 (the evaporation-temperature-elevation relations established for the stations observed in Texas and New Mexico), used in conjunction with the observed mean monthly temperatures at Lake Conchos.

The elevation of Lake Conchos is 4 320 ft. above sea level. On Diagram II of Fig. 13 is drawn a line at that elevation from which were read off the mean monthly temperatures necessary at that elevation to cause monthly evaporation depths (from 3-ft. square floating-pans) of 2, 4, 6, 8, and 10 in. Those five values of mean monthly temperature are plotted on Fig. 15, where it is seen that they conform almost exactly to a straight line. This straight line (which represents

the monthly evaporation depths corresponding to all monthly mean temperatures at places on the Great Plateau in Texas and New Mexico having elevations of 4 320 ft.) it is assumed may be applied safely to Lake Conchos; also on the Great Plateau, but about 500 miles farther south, in Mexico.

In Table 37 and on Fig. 14 are given the observed monthly mean temperatures (the mean of the daily mean temperatures for each



Note:-Later data give the following additional monthly mean temperatures at La Boquilla

Jan. 1914-Max., 69°; Min., 55°; Mean, 62.°
Feb. 1914- " 73°; " 40°; " 56.°

The corresponding monthly mean temperatures at El Paso were

Jan. 1914-Max., 61°; Min., 35°; Mean, 48.°
Feb. 1914- " 61.5°; " 36.5°; " 49.°

FIG. 14.

month) at La Boquilla for 21 months, April, 1912, to December, 1913, inclusive. In Table 30 these twenty-one monthly mean temperatures are repeated (with that for January, 1914, added), and there are given also the twenty-two corresponding depths of monthly evaporation, as read off from Fig. 15.

From Table 30, the evaporation depth from 3-ft. square pans floating on Lake Conchos should have been 80.43 in. for the calendar year,

1913; but, for a composite year, made up by adding the mean evaporations for each of the 12 months, it would be 85.59 in., or 6.5% greater.

From the 22 months' record, however, it is practicable to make up eleven periods, of 12 consecutive months each, and so gain much more knowledge of the variations in yearly evaporation depth. This is done in Table 31.

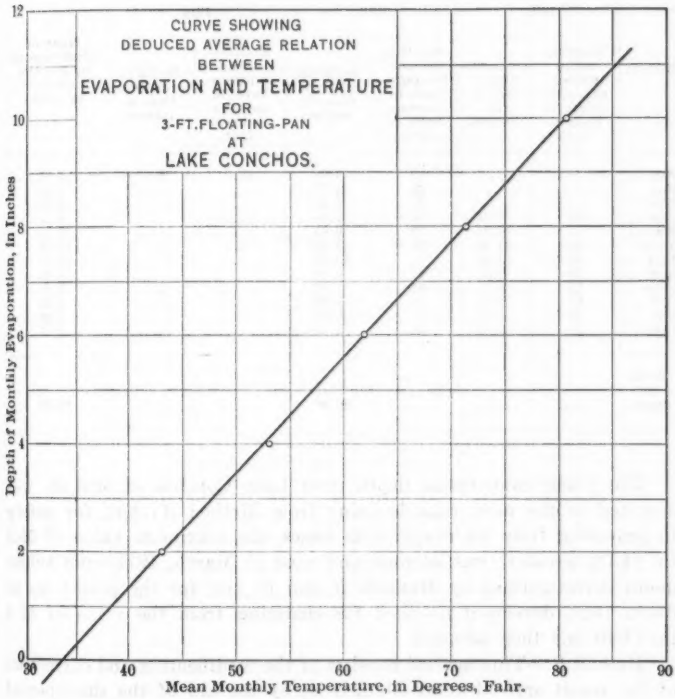


FIG. 15.

The variations in the evaporation depths for 12-month periods are shown better by Table 32.

It is believed that the most reliable and accurate value from Method *A* of the yearly evaporation depth from 3-ft. square pans floating on Lake Conchos is (so far as can be obtained from the data) 84.74 in.

By Subsidiary Conclusion (*c*), the yearly evaporation depth from a lake surface is only 62% of that from a 3-ft. square pan floating

thereon;* hence the corresponding yearly evaporation depths from Lake Conchos itself, by Method *A*, would be as given in Table 33.

TABLE 30.—OBSERVED MONTHLY MEAN TEMPERATURES; AND CORRESPONDING DEPTHS OF EVAPORATION, AS DEDUCED FOR 3-FT. SQUARE FLOATING-PANS, AT LAKE CONCHOS.

Month.	1912.		1913.		1914.		Mean of observations by months. Evaporation, in inches.
	Monthly mean temperature, in degrees, Fahrenheit.	Deduced evaporation, in inches.	Monthly mean temperature, in degrees, Fahrenheit.	Deduced evaporation, in inches.	Monthly mean temperature, in degrees, Fahrenheit.	Deduced evaporation, in inches.	
Jan....	.....	.....	52	3.90	62.0	6.00	4.95
Feb....	.....	.....	56	4.75	.....	.....	4.75
Mar....	.....	.....	63	6.23	.....	.....	6.23
Apr....	73	8.34	71	7.91	.....	.....	8.13
May...	82	10.24	79	9.61	.....	.....	9.93
June...	84	10.65	79	9.61	.....	.....	10.13
July....	85	10.86	81	10.03	.....	.....	10.44
Aug ...	79	9.61	77	9.18	.....	.....	9.40
Sept...	75	8.75	67	7.07	.....	.....	7.91
Oct....	67	7.07	60	5.59	.....	.....	6.33
Nov....	55	4.54	52	3.90	.....	.....	4.22
Dec....	51	3.69	46	2.65	.....	.....	3.17
Total or mean.	.....	.....	.....	80.43	.....	.....	85.59

The yearly evaporation depth from Lake Conchos of 52.5 in. was accepted as the most reliable value from Method *A*; but, for safety in providing fully for evaporation losses, the maximum value of 55.0 in. (4.7% greater) was adopted and used in March, 1914—and subsequent investigations by Methods *B* and *C*, and for the period up to June, 1914, developed no need for changing from the value of 55.0 in. (1.40 m.) then adopted.

Method B.—This method consists of the modification and correction of the result arrived at by Method *A*, by the aid of the short-period pan measurements of evaporation made at Lake Conchos, first for 2 and later for 5 months. Two variants of Method *B* were used, B_a and B_b; and a third variant B_c was attempted, but was rejected as unreliable.

The rejected method B_c consisted of a direct percentage comparison of simultaneous observations of pan evaporations at El Paso and at Lake Conchos for the same short period (16 days), used in conjunction with the observed yearly evaporation depth at El Paso.

* See Table 2.

TABLE 31.—ESTIMATED DEPTH OF EVAPORATION AT LAKE CONCHOS FOR ELEVEN 12-MONTH PERIODS. From Evaporation-Temperature Study.

Month.	ESTIMATED DEPTHS OF EVAPORATION BY 12-MONTH PERIODS (IN 3-FT. SQUARE FLOATING-PANS).											Means.	
	Apr., 1912, to Mar., 1913, inclusive.	May, 1913, to Apr., 1913, inclusive.	June, 1912, to May, 1913, inclusive.	July, 1912, to June, 1913, inclusive.	Aug., 1912, to July, 1913, inclusive.	Sept., 1912, to Aug., 1913, inclusive.	Oct., 1912, to Sept., 1913, inclusive.	Nov., 1912, to Oct., 1913, inclusive.	Dec., 1912, to Nov., 1913, inclusive.	Jan., 1913, to Dec., 1913, inclusive.	Feb., 1913, to Jan., 1914, inclusive.	In inches.	Percentage of distribution.
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.		
January...	3.90	3.90	3.90	3.90	3.90	3.90	3.90	3.90	3.90	3.90	6.00	4.09	4.8%
February..	4.75	4.75	4.75	4.75	4.75	4.75	4.75	4.75	4.75	4.75	4.75	4.75	5.6
March.....	6.23	6.23	6.23	6.23	6.23	6.23	6.23	6.23	6.23	6.23	6.23	6.23	7.4
April.....	8.34	7.91	7.91	7.91	7.91	7.91	7.91	7.91	7.91	7.91	7.91	7.95	9.4
May.....	10.24	10.24	9.61	9.61	9.61	9.61	9.61	9.61	9.61	9.61	9.61	9.72	11.5
June.....	10.65	10.65	10.65	9.61	9.61	9.61	9.61	9.61	9.61	9.61	9.61	9.90	11.7
July.....	10.86	10.86	10.86	10.86	10.03	10.03	10.03	10.03	10.03	10.03	10.03	10.33	12.2
August....	9.61	9.61	9.61	9.61	9.61	9.18	9.18	9.18	9.18	9.18	9.18	9.38	11.1
September	8.75	8.75	8.75	8.75	8.75	8.75	7.07	7.07	7.07	7.07	7.07	7.99	9.4
October...	7.07	7.07	7.07	7.07	7.07	7.07	7.07	5.59	5.59	5.59	5.59	6.53	7.7
November.	4.54	4.54	4.54	4.54	4.54	4.54	4.54	4.54	3.90	3.90	3.90	4.37	5.1
December.	3.69	3.69	3.69	3.69	3.69	3.69	3.69	3.69	3.69	2.65	2.65	3.50	4.1
Totals.....	88.63	88.20	87.57	86.53	85.70	85.27	83.59	82.11	81.47	80.43	82.53	84.74	100%

TABLE 32.—VARIATIONS IN YEARLY EVAPORATION DEPTH.

12-month period, or year.	Corresponding Depth of Evaporation from 3-ft. Square Pans Floating on Lake Conchos.	
	In inches.	Percentages.
April, 1912, to March, 1913, inclusive	88.63	104.7
May, " " April, " "	88.20	104.0
June, " " May, " "	87.57	103.3
July, " " June, " "	86.53	102.0
August, " " July, " "	85.70	101.0
September, " " August, " "	85.27	100.6
October, " " September, " "	83.59	98.6
November, " " October, " "	82.11	96.9
December, " " November, " "	81.47	96.0
January, 1913, " December, " "	80.43	94.9
February, " " January, 1914, "	82.53	97.4
Average for 11 periods	84.74	100
Extremes for 11 periods	88.63 and 80.43	+ 4.7 and − 5.1
Corresponding composite value from Table 31	85.59	101.0

TABLE 33.—YEARLY EVAPORATION DEPTHS FROM LAKE CONCHOS, BY METHOD *A*.

	Pan evaporation.	Lake evaporation.
Maximum value	88.63 × 0.62	= 55.0 in. = 104.7%
Minimum value	80.43 × "	= 49.8 " = 94.9
Value for calendar year, 1913	80.43 × "	= 49.8 " = 94.9
Value for composite year, all mean monthly temperatures	85.59 × "	= 53.0 " = 101.0
Value as average for the eleven 12-month periods	84.74 × "	= 52.5 " = 100.0

The observed pan evaporations and mean temperatures at El Paso, and at La Boquilla, for January-March, 1914, are given in full in Table 34 (Plate LXVI), and are summarized in Table 35.

The observations at El Paso for January and February were rejected, because they covered only 3 days in January and 2 days in February. For various reasons, including shooting of the pans by Mexicans and cowboys, it was found impracticable to carry out a fairly unbroken series of evaporation observations at El Paso within the time available.

As shown by Table 35, the deduced evaporation depths at El Paso from 3-ft. square floating-pans for the entire month of March were 2.43 in., as measured in Pan No. 1, and 2.53 in., as measured in Pan No. 2, or a mean depth of 2.48 in.; the corresponding evaporation depths at Lake Conchos for March were 4.40, 5.12, 5.44, and 5.74 in., as measured in four pans, or a mean of 5.17 in. Hence the evaporation

TABLE 34.—Measured Evaporatio

Date, 1914. January.	At La Boquilla, Mexico.								El Paso, Texas.						Date, 1914. February.	At La Boquilla, Mexico.							
	Evaporation, in inches.				Temperature.			Weather conditions at La Boquilla.	Evaporation, in inches.			Temperature.				Evaporation, in inches.				Temperature.			Wea
	Land-pans.		Floating-pans.						Land-pans.							Land-pans.		Floating-pans.					
	No. 1.	No. 3.	No. 2.	No. 4.	Maximum, degrees, Fahr.	Minimum, degrees, Fahr.	Mean, degrees, Fahr.		No. 1.	No. 2.	No. 3.	Maximum, degrees, Fahr.	Minimum, degrees, Fahr.	Mean, degrees, Fahr.		No. 1.	No. 3.	No. 2.	No. 4.	Maximum, degrees, Fahr.	Minimum, degrees, Fahr.	Mean, degrees, Fahr.	
1	....	....	....	....	71	..	..		....	....	....	69	42	55	1	4/32	4/32	6/32	....	71	28	49	Bright and clea
2	....	....	....	....	71	..	..		....	....	....	62	39	51	2	5/32	3/32	16/32	....	74	29	52	Morning cloud
3	....	....	....	....	71	..	..		....	....	....	68	35	51	3	5/32	4/32	6/32	....	79	40	59	Bright and clea
4	....	....	....	....	71	..	..		....	....	....	56	39	47	4	†	6/32	6/32	....	77	40	58	Morning bright
5	....	....	....	....	71	..	..		....	....	....	59	31	45	5	9/32	†	*	....	75	45	60	Bright, clear, w
6	....	....	....	....	65	..	..		....	....	....	61	31	46	6	10/32	†	*	....	73	53	62	A. M. cloudy, a
7	....	....	....	....	61	..	..		....	....	....	62	33	47	7	7/32	†	*	....	75	36	56	Cloudy, light b
8	....	....	....	....	71	..	..		....	....	....	72	37	55	8	6/32	6/32	8/32	4/32	62	45	54	Cloudy, light b
9	....	....	....	....	66	..	..		....	....	....	68	42	55	9	4/32	4/32	6/32	4/32	73	37	55	Bright and clea
10	....	....	....	....	56	..	..		....	....	....	51	33	42	10	6/32	6/32	4/32	4/32	75	40	57	Bright and clea
11	....	....	....	....	59	..	..		....	....	....	45	35	40	11	6/32	6/32	4/32	4/32	82	46	64	Morning bright
12	....	....	....	....	60	..	..	Clear bright day	....	....	....	52	35	43	12	6/32	4/32	7/32	*	76	45	60	Morning cloud
13	4/32	....	....	....	67	..	..	Clear bright day	....	....	....	59	26	43	13	6/32	4/32	7/32	*	75	40	58	Morning bright
14	4/32	4/32	....	....	71	63	67	Clear bright day	....	....	....	62	28	46	14	6/32	6/32	5/32	*	74	42	58	Bright and clea
15	6/32	10/32	....	....	75	64	69	Clear bright day, warm	....	....	....	71	31	51	15	6/32	6/32	4/32	*	68	40	54	Bright, clear, w
16	7/32	8/32	....	....	78	63	68	Morning clear, cloudy afternoon	....	....	....	74	41	57	16	6/32	6/32	4/32	*	70	45	57	Cloudy, high w
17	9/32	6/32	2/32	....	76	69	73	Morning cloudy ; bright, clear afternoon	....	....	....	61	46	54	17	2/32	3/32	5/32	*	71	48	60	Cloudy, 1.5 mm
18	6/32	6/32	4/32	....	72	62	67	Clear and bright, stiff breeze	....	....	....	68	40	54	18	3/32	3/32	6/32	*	72	40	56	Bright and clea
19	10/32	6/32	6/32	....	78	64	68	Clear and bright, moderate breeze	....	....	....	67	37	52	19	2/32	4/32	4/32	*	72	44	58	Bright and clea
20	6/32	6/32	*	....	67	54	61	Cloudy, high winds	....	....	....	58	39	48	20	7/32	5/32	6/32	2/32	70	45	57	Bright and clea
21	6/32	6/32	*	....	68	55	59	Morning cloudy, afternoon clear, no wind	....	....	....	62	28	45	21	7/32	5/32	6/32	2/32	80	38	59	Bright and clea
22	6/32	6/32	*	....	67	57	62	Clear and bright, no wind	....	....	....	64	37	46	22	7/32	7/32	*	*	74	43	58	Cloudy, very h
23	6/32	8/32	4/32	....	72	61	66	Cloudy, high wind	....	....	....	56	41	48	23	8/32	6/32	*	*	75	44	60	Morning cloud
24	14/32	8/32	6/32	....	77	60	69	Bright and clear, calm	....	....	....	63	43	53	24	8/32	6/32	*	*	70	33	51	Morning cloud
25	8/32	6/32	6/32	....	69	61	65	Bright and clear, calm	4/32	4/32	4/32	64	45	55	25	7/32	7/32	*	*	60	22	41	Bright and clea
26	6/32	8/32	8/32	....	79	68	73	Clear, light breeze	4/32	4/32	3/32	76	39	57	26	6/32	....	8/32	*	78	22	50	Bright and clea
27	10/32	5/32	*	6/32	85	69	77	Bright and clear, light breeze	4/32	3/32	3/32	73	54	64	27	6/32	....	8/32	*	70	46	58	Morning bright
28	20/32	15/32	*	*	78	49	64	Bright and clear, very high wind	Discontinued.			54	32	43	28	5/32	....	7/32	*	65	30	47	Morning bright
29	16/32	12/32	*	*	57	29	43	Bright and clear, bush wind				41	22	31									
30	4/32	5/32	6/32	8/32	54	30	42	Bright and clear, light wind	....	....	....	49	20	35	Total or average.	5.1	3.5	4.2	0.63	73	40	56	.....
31	4/32	4/32	4/32	....	65	49	57	Bright and clear, calm	....	....	....	59	25	42									
Total or average.	4.87	4.04	1.44	0.44	69	57	64	.....	0.37	0.34	0.31	61.4	35.4	48.3									

* Submerged.

† Pan being reset.

TABLE 34.—MEASURED EVAPORATIONS AND TEMPERATURES AT LA BOQUILLA, MEXICO, AND EL PASO, TEX., FOR JANUARY, FEBRUARY, AND MARCH, 1914.

AT LA BOQUILLA, MEXICO.							EL PASO, TEXAS.						Date, 1914, March.	AT LA BOQUILLA, MEXICO.							
tion, in inches.			Temperature.			Weather conditions at La Boquilla.	Evaporation, in inches.			Temperature.				Evaporation, in inches.				Temperature.			
	Floating-pans.						Land-pans.							Land-pans.		Floating-pans.					
3.	No. 2.	No. 4.	Maximum, degrees, Fahr.	Minimum, degrees, Fahr.	Mean, degrees, Fahr.		No. 1.	No. 2.	No. 3.	Maximum, degrees, Fahr.	Minimum, degrees, Fahr.	Mean, degrees, Fahr.		No. 1.	No. 3.	No. 2.	No. 4.	Maximum, degrees, Fahr.	Minimum, degrees, Fahr.	Mean, degrees, Fahr.	
2	6/32	....	71	28	49	Bright and clear, calm	....	....	(Discontinued.)	67	30	48	1	6/32	6/32	7/32	....	76	34	55	Brig
2	16/32	....	74	29	52	Morning cloudy; afternoon bright, clear	....	....		66	40	53	2	5/32	7/32	6/32	....	76	36	56	Brig
2	6/32	....	79	40	59	Bright and clear, perceptible breeze	....	....		64	39	52	3	6/32	8/32	‡	....	69	50	60	Brig
2	6/32	....	77	40	58	Morning bright, clear; afternoon cloudy	....	....		63	43	53	4	6/32	7/32	‡	*	60	31	45	Brig
	*	....	75	45	60	Bright, clear, very high wind	....	....		66	42	54	5	6/32	7/32	‡	*	73	28	51	Brig
	*	....	73	52	63	A. M. cloudy, afternoon bright and clear, very high wind	....	....		58	39	48	6	6/32	8/32	‡	*	72	38	55	Brig
	*	....	75	38	56	Cloudy, light breeze	....	....		57	29	43	7	6/32	8/32	‡	*	68	38	53	Brig
2	8/32	4/32	62	45	54	Cloudy, light breeze	....	....		59	31	45	8	5/32	6/32	4/32	6/32	72	34	53	Brig
2	6/32	4/32	73	37	55	Bright and clear, perceptible breeze	....	....		65	30	47	9	5/32	6/32	4/32	6/32	82	42	62	Clou
2	4/32	4/32	75	40	57	Bright and clear, perceptible breeze	....	....		67	35	51	10	....	....	....	....	..	..	..	.....
2	4/32	4/32	82	46	64	Morning bright, clear; afternoon cloudy	....	....		71	36	54	11	....	....	....	....	..	..	..	.....
2	7/32	*	76	45	60	Morning cloudy; afternoon bright and clear, high wind	....	....		68	48	58	12	....	....	....	....	..	..	..	.....
2	7/32	*	75	40	58	Morning bright and clear; afternoon cloudy, light wind	....	....		60	33	46	13	....	....	....	....	..	..	..	.....
2	5/32	*	74	42	58	Bright and clear, high wind	....	....		61	33	47	14	....	....	....	....	..	..	..	.....
2	4/32	*	68	40	54	Bright, clear, moderate breeze	....	....		58	33	46	15	....	....	....	....	..	..	..	.....
2	4/32	*	70	45	57	Cloudy, high wind	....	....		60	40	50	16	....	....	....	....	..	..	..	.....
2	5/32	*	71	48	60	Cloudy, 1.5 mm. rain, perceptible breeze	....	....		59	42	51	17								
2	6/32	*	72	40	56	Bright and clear, high wind	....	....		59	45	52	18								
2	4/32	*	72	44	58	Bright and clear, light wind	....	....		63	45	54	19								
2	6/32	2/32	70	45	57	Bright and clear, light wind	....	....		67	42	54	20								
2	6/32	2/32	80	38	59	Bright and clear, perceptible wind	....	....		72	40	56	21								
	*	*	74	43	58	Cloudy, very high wind	....	....		55	42	48	22								
	*	*	75	44	60	Morning cloudy; afternoon bright, high wind	....	....		57	35	46	23								
	*	*	70	33	51	Morning cloudy; afternoon bright, high wind	....	....		42	26	34	24								
	*	*	60	22	41	Bright and clear, light wind	....	....		54	26	40	25								
	8/32	*	78	22	50	Bright and clear, moderate breeze	....	....		64	32	48	26								
	8/32	*	70	46	58	Morning bright and clear; afternoon cloudy, high wind	2/32	4/32		54	32	43	27								
	7/32	*	65	30	47	Morning bright and clear; moderate breeze	2/32	2/32		66	34	50	28								
													29								
													30								
5	4.2	0.63	73	40	56		0.12	0.19		61.5	36.5	49	31								
													Total or average.	1.60	1.97	0.66	0.37	....	....	....	.....

† Pan being reset. * Submerged. ‡ Pan leaking.

Mexico.	El Paso, Texas.					
Weather conditions at La Boquilla.	Evaporation, in inches.			Temperature.		
	Land-pans.			Maximum, degrees, Fahr.	Minimum, degrees, Fahr.	Mean, degrees, Fahr.
	No. 1.	No. 2.	No. 3.			
Bright and clear, brisk breeze	4/32	6/32	(Discontinued.)	72	26	54
Bright and clear, stiff breeze	2/32	2/32		72	42	57
Bright and clear, high wind	2/32	2/32		54	34	44
Bright and clear, stiff wind	2/32	2/32		62	33	47
Bright and clear, squally breeze	4/32	4/32		71	43	57
Bright and clear, high wind	2/32	2/32		67	37	52
Bright and clear, high wind	4/32	4/32		58	36	47
Bright and clear, brisk breeze	4/32	4/32		68	31	49
Cloudy, light breeze	4/32	4/32		72	39	56
.....	4/32	2/32		74	44	59
.....	2/32	4/32		53	36	45
.....	4/32	2/32		57	32	44
.....	2/32	4/32		64	28	46
.....	4/32	4/32		73	35	54
.....	2/32	2/32				
.....	4/32	4/32				
.....	1.56	1.63				

*Submerged.

NOTES.

Evaporation-Pans at La Boquilla.

The evaporation-pans set at La Boquilla are of galvanized iron, 3 ft. square and 1 ft. deep. A rain-gauge was set near each group of pans.

The land-pans were originally set on top of the ground (or rock) surface and a strip of canvas was tacked around the outside of the pans to prevent the sun from striking their metal sides. On February 4th the land-pans were reset in earth. Because of the rocky character of the ground it was necessary to haul the soil to the site and deposit it around the pans. Secure fences were provided around the pans as protection from animals, and an observer was commissioned to ride continually back and forth to the pans to prevent any interference with them, observe any leaks or unusual conditions of any nature, and to make readings.

The floating-pans were protected from spray by flash-boards on the rafts, but their records were frequently interfered with by leaks or waves, and by being blown ashore during high winds.

The description of the location of individual pans follows:

Pan No. 1 (*Land-Pan*).—Set January 12th, 1914, at 5.30 P. M., about 2 000 ft. up stream from La Boquilla Dam at Elevation 1 296 m. Pan is in path of prevailing winds, and gets sun entire day.

Pan No. 2 (*Floating-Pan*).—Set January 14th, 1914, at 5.40 P. M., floating on Lake Conchos near Land-Pan No. 1. Pan is built inside a strong raft which has flash-boards to prevent spray from entering pan.

Pan No. 3 (*Land-Pan*).—Set January 14th, 1914, at 4 P. M., on a promontory about 4 km. up stream from La Boquilla Dam, at Elevation 1 317 m. Pan is in open stretch of country.

Pan No. 4 (*Floating-Pan*).—Set January 20th, 1914, floating on Lake Conchos near Land-Pan No. 3. Pan is of similar construction, and protected in a similar manner as No. 2, but winds and waves interfered with the record to a great extent.

Evaporation-Pans at El Paso.

Three floating-pans and three land-pans 3 ft. square, of the same type as those set at La Boquilla, were set at a cement plant about 3 miles from El Paso, Tex., on January 24th, 1914. The reservoir in which the floating-pans were set was emptied immediately after their installation so that no record was secured. The land-pans, after but 3 days' record was secured, were so interfered with by winds, which blew sand into them, that two pans were moved from their original positions into a rear yard near the Weather Bureau Station in El Paso, on February 26th. It is not known to what extent near-by buildings or fences may interfere with air currents, or tend toward a reduced evaporation depth at this location.

The El Paso records are not considered satisfactory.

Temperatures.

Temperatures given for La Boquilla are those observed by the Mexican Northern Power Company with maximum and minimum thermometers. During January, 1914, the minimum thermometer was observed to be inexact, and the temperatures here given for that month are those observed at Pilar de Conchos at the up-stream end of the reservoir. The temperatures at Pilar de Conchos are known to conform very closely to those at La Boquilla.

The record of temperature for El Paso here given is that taken by the United States Weather Bureau.

The mean daily temperature as here used is the mean of the maximum and minimum for the day, as is the practice of the United States Weather Bureau. The mean monthly temperature is the average of the mean daily temperatures for the month.

TABLE 35.—Observed Pan Evaporations and Mean Temperatures, January–March, 1914.

At Lake Conchos, Mexico.

No. of evaporation pan.	(Land-pan) No. 1.			(Floating-pan) No. 2.			(Land-pan) No. 3.			(Floating-pan) No. 4.		
For month of (1914)	Jan.	Feb.	Mar.	Jan.	Feb.	Mar.	Jan.	Feb.	Mar.	Jan.	Feb.	Mar
No. of days measured	19	27	9	9	21	4	18	22	9	2	6	2
Total depth measured, in inches	4.87	5.1	1.60	1.44	4.2	0.66	4.04	3.5	1.97	0.44	0.63	0.37
Average daily evaporation depth, in inches	0.26	0.19	0.18	0.16	0.20	0.16	0.22	0.16	0.22	0.22	0.10	0.19
Corresponding depth for full month, in inches	7.95	5.3	5.5	4.96	5.60	5.12	6.96	4.46	6.8	6.82	2.94	5.74
Depth for 3-ft floating-pan,* in inches	6.36	4.25	4.40	4.96	5.60	5.12	5.57	3.57	5.44	6.82	2.94	5.74
Mean temperature, in degrees, Fahrenheit, for period of measurement.	64	56	54	64.5	56	56	64	56	54	60	58	57
Mean temperature, in degrees, Fahrenheit, for entire month	62	56	.....	62	56	.....	62	56	.....	62	56	.....

At El Paso, Tex.

No. of evaporation pan.	(Land-pan) No. 1.			(Land-pan) No 2.			(Land-pan) No. 3.		
For month of (1914)	Jan.	Feb.	Mar.	Jan.	Feb	Mar.	Jan.	Feb.	Mar.
Number of days measured	3	2	16	3	2	16	3	.....	.....
Total depth measured, in inches	0.37	0.12	1.56	0.34	0.19	1.63	0.31	.....	.....
Average daily evaporation depth, in inches	0.12	0.06	0.10	0.11	0.10	0.10	0.10	.....	.....
Corresponding depth for full month, in inches	3.82	1.68	3.03	3.51	2.66	3.16	3.20	.....	.....
Depth for 3-ft. floating-pan,* in inches	3.05	1.34	2.43	2.81	2.13	2.53	2.56	.....	.....
Mean temperature, in degrees, Fahrenheit, for period of measurement.	59	28	†51	59	27	†51	59	.....	.....
Mean temperature, in degrees, Fahrenheit, for entire month	48	49	.....	48	49	.....	48	.....	.....

* As 80% of land pan.

† These mean temperature results cover only 14 days of the period of measurement. Temperatures recorded by U. S. Weather Bureau.

depth at Lake Conchos apparently was $(5.17 \div 2.48) = 2.09$ times that at El Paso.

TABLE 37.—MEAN MONTHLY TEMPERATURES AT EL PASO, TEX., AND LA BOQUILLA, MEXICO.

In Degrees, Fahrenheit.

Station and Month.	EL PASO, TEX.			LA BOQUILLA, MEXICO.		
	Maximum.	Minimum.	Mean.	Maximum.	Minimum.	Mean.
1912, April	72.0	46.5	59.0	85.0	....	*73.0
May	85.5	57.5	71.5	95.0	....	*82.0
June	90.0	65.0	78.0	97.0	....	*84.0
July	93.0	69.0	81.0	98.0	....	*85.0
August	89.0	67.0	78.0	92.0	67.0	79.0
September	82.5	60.0	71.0	88.0	62.0	75.0
October	74.0	50.5	62.0	81.0	54.0	67.0
November	62.0	36.0	49.0	69.0	41.0	55.0
December	50.0	30.0	40.0	64.0	39.0	51.0
1913, January	53.0	28.0	40.0	67.0	37.0	52.0
February	57.0	35.0	46.0	68.0	45.0	56.0
March	65.0	39.0	52.0	76.0	51.0	63.0
April	75.0	47.0	61.0	82.0	61.0	71.0
May	85.5	58.0	72.0	93.0	66.0	79.0
June	89.0	64.0	77.0	93.0	65.0	79.0
July	94.0	69.0	81.5	93.0	70.0	81.0
August	90.0	67.0	79.0	89.0	65.0	77.0
September	81.0	58.0	69.0	79.0	....	*67.0
October	77.0	51.0	64.0	73.0	....	*60.0
November	66.0	44.0	55.0	65.0	....	*52.0
December	52.0	32.0	42.0	58.0	....	*46.0

NOTE.—The maximum and minimum temperatures given in Table 37 are the means of all the maximum and all the minimum daily observations for each month. The mean temperatures given are the means of all daily means for the month, the latter being the mean of the maximum and minimum daily.

*The mean annual temperatures for 1913 were—for El Paso, 61.5°; for La Boquilla, 65.2°. Mean monthly temperatures at La Boquilla shown thus (*73°) are deduced from observed maximum temperatures for the corresponding months. During these months no minimum temperatures were observed, due to the fact that no recording thermometer for minimum temperatures was on hand. The mean temperatures, therefore, were deduced by two methods: (1) by percentage relations between maximum and mean temperatures during months when measurements were made, and (2) by first deducing minimum temperatures by average daily differences between maximum and minimum temperatures for observed months.

The daily temperatures at La Boquilla were taken from the monthly hydrographic report sheets of the Mexican Northern Power Co., Ltd., and those at El Paso from the U. S. Weather Bureau Climatological Reports.

The observed evaporation depth at El Paso for a year is about 82.0 in. (see Table 17, Plate LXII); hence, on the supposition that the simultaneous observations of March, 1914, represent the true relative evaporation depths at El Paso and Lake Conchos, the yearly evaporation depth at the latter from 3-ft. square floating-pans should be $(82.0 \times 2.09) = 171$ in., or 14.25 ft.

Such an evaporation depth evidently is absurd; and an analysis of the El Paso data in connection with this method of deduction (B_o) shows that the method is unreliable and unsafe when used with short-period observations, for the reason that during the months of small

TABLE 36.—MEASURED EVAPORATIONS AND TEMPERATURES

Date, 1914, March.	Evaporation, in Inches.				Temperature.			Weather conditions.
	Land-pans.		Floating-pans.					
	No. 1.	No. 3.	No. 2.	No. 4.	Maximum, degrees, Fahr.	Minimum, degrees, Fahr.	Mean, degrees, Fahr.	
1	6/32	6/32	7/32	†	76	34	55	Bright and clear, brisk breeze
2	5/32	7/32	6/32	†	76	36	56	Bright and clear, stiff breeze
3	6/32	8/32	*	†	69	50	60	Bright and clear, high wind
4	6/32	7/32	*	†	60	31	45	Bright and clear, stiff wind
5	6/32	7/32	*	‡	73	29	51	Bright and clear, squally breeze
6	6/32	8/32	*	‡	72	38	55	Bright and clear, high wind
7	6/32	8/32	*	‡	68	38	53	Bright and clear, high wind
8	5/32	6/32	4/32	6/32	72	34	53	Bright and clear, brisk breeze
9	5/32	6/32	4/32	6/32	82	42	62	Cloudy, light breeze
10	....	....	....	†	83	50	67	Bright and clear, brisk wind
11	....	....	....	†	70	45	57	Bright and clear, high wind
12	22/32	22/32	20/32	†	63	40	52	Cloudy, brisk wind
13	....	....	....	†	67	33	50	Bright and clear, brisk wind
14	12/32	14/32	12/32	†	80	37	53	Bright and clear, light wind
15	....	....	....	†	80	40	60	Bright and clear, moderate wind, trace rain
16	....	....	....	†	79	51	65	Bright and clear, brisk wind
17	18/32	14/32	16/32	†	82	43	63	Bright and clear, brisk wind
18	....	....	....	†	78	52	65	Cloudy, very high wind, 1 mm. rain
19	....	....	....	†	44	39	41	Cloudy, high wind, 2.7 cm. rain
20	....	....	....	†	52	34	43	Cloudy, light wind, trace rain
21	26/32	24/32	22/32	†	67	40	54	Cloudy, brisk breeze
22	....	....	....	†	66	42	54	Bright and clear, moderate wind
23	6/32	6/32	6/32	†	79	43	61	Bright and clear, calm
24	....	....	....	†	77	45	61	Bright and clear, brisk wind
25	10/32	8/32	8/32	†	76	47	61	Bright and clear, moderate wind
26	....	....	....	†	81	48	65	Bright and clear, moderate breeze
27	12/32	10/32	10/32	†	84	55	69	Bright and clear, moderate breeze
28	....	....	†	†	80	50	65	Bright and clear, high wind
29	....	....	†	†	83	55	69	Cloudy, high wind
30	....	....	†	†	83	52	68	Bright and clear, moderate breeze
31	44/32	44/32	†	†	80	49	64	.....
Total or average.	6.28	6.41	3.60	0.38	74	43	58	.....

* Pan leaking. † No record available. ‡ Pan submerged.

Date, 1914, April.	Evaporation, in Inches.				Temperature.			Weather conditions.
	Land-pans.		Floating-pans.					
	No. 1.	No. 3.	No. 2.	No. 4.	Maximum, degrees, Fahr.	Minimum, degrees, Fahr.	Mean, degrees, Fahr.	
1	12/32	10/32	‡	†	90	52	71	Bright and clear, perceptible wind
2	....	....	‡	†	86	55	70	Bright and clear, perceptible wind
3	10/32	8/32	‡	†	86	52	69	Bright and clear, moderate breeze
4	....	....	....	†	79	52	66	Bright and clear, moderate breeze
5	....	....	....	†	85	54	69	Bright and clear, high wind
6	14/32	16/32	10/32	†	90	61	76	Cloudy, high wind
7	....	....	....	†	87	57	72	Bright and clear, high wind
8	38/32	36/32	30/32	†	82	55	68	Bright and clear, high wind
9	....	....	....	†	75	42	59	Bright and clear, moderate breeze
10	....	....	....	†	83	59	71	Bright and clear, high wind
11	....	....	....	†	89	57	73	Bright and clear, high wind
12	....	....	....	†	81	57	69	Bright and clear, calm
13	....	....	....	†	81	50	66	Bright and clear, moderate breeze
14	64/32	60/32	56/32	†	88	48	68	Bright and clear, calm
15	....	....	....	†	95	54	74	Bright and clear, perceptible breeze
16	28/32	24/32	14/32	†	94	64	79	Bright and clear, high wind
17	....	....	....	†	92	64	78	Cloudy, high wind
18	30/32	26/32	28/32	†	88	52	70	Hazy, moderate breeze
19	....	....	....	†	88	50	69	Bright and clear, moderate breeze
20	22/32	20/32	20/32	†	93	50	71	Cloudy, moderate breeze
21	....	....	....	†	90	62	76	Bright, clear A. M.; cloudy P. M.; high wind
22	....	....	....	†	93	63	78	Bright, clear A. M.; cloudy P. M.; trace rain
23	....	....	....	†	95	59	77	Bright and clear, high wind
24	....	....	....	†	91	50	70	Bright and clear, high wind
25	44/32	28/32	42/32	†	91	63	77	Cloudy, high wind
26	....	....	....	†	93	57	75	Hazy, moderate breeze
27	32/32	32/32	32/32	†	93	57	77	Hazy, calm
28	....	....	....	†	90	57	74	Cloudy, calm
29	24/32	32/32	....	†	99	54	76	Hazy, calm
30	14/32	16/32	47/32	†	93	54	76	Clear, perceptible breeze
Total or average.	10.38	9.63	8.72	†	89	55	72	.....

† No record available. ‡ Pan submerged.

Date, 1914, May.
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Total or averag

TABLE 36.—MEASURED EVAPORATIONS AND TEMPERATURES AT LA BOQUILLA, MEXICO, FOR MARCH, APRIL, MAY, AND JUNE, 1914.

Temperature.			Weather conditions.	Date, 1914, May.	Evaporation, in Inches.				Temperature.			Weather conditions.
					Land-pans.		Floating-pans.					
Maximum, degrees, Fahr.	Minimum, degrees, Fahr.	Mean, degrees, Fahr.			No. 1.	No. 3.	No. 2.	No. 4.	Maximum, degrees, Fahr.	Minimum, degrees, Fahr.	Mean, degrees, Fahr.	
90	52	71	Bright and clear, perceptible wind	1	....	....	....	†	93	59	76	Clear, perceptible breeze
86	55	70	Bright and clear, perceptible wind	2	....	....	....	†	98	59	79	Cloudy, calm
86	52	69	Bright and clear, moderate breeze	3	42/32	48/32	47/32	†	95	57	76	Bright and clear, high wind
79	52	66	Bright and clear, moderate breeze	4	....	....	....	†	96	52	74	Bright and clear, high wind
85	54	69	Bright and clear, high wind	5	....	....	....	†	93	52	72	Bright and clear, moderate breeze
90	61	76	Cloudy, high wind	6	40/32	48/32	38/32	†	98	57	77	Bright and clear, light wind
87	57	72	Bright and clear, high wind	7	....	....	....	†	98	59	79	Bright and clear, calm
82	55	68	Bright and clear, high wind	8	34/32	32/32	30/32	†	96	61	79	Bright and clear, moderate breeze
75	42	59	Bright and clear, moderate breeze	9	....	†	....	†	100	66	83	Bright and clear, moderate breeze
83	59	71	Bright and clear, high wind	10	34/32	†	30/32	†	104	65	84	Bright and clear, calm
89	57	73	Bright and clear, high wind	11	16/32	10/32	12/32	†	100	66	83	Hazy A. M., bright P. M.; light breeze
81	57	69	Bright and clear, calm	12	....	....	....	†	99	71	85	....
81	50	66	Bright and clear, moderate breeze	13	30/32	32/32	28/32	†	94	66	80	Cloudy, moderate wind, trace rain
88	48	68	Bright and clear, calm	14	....	....	....	†	83	59	71	Bright and clear, moderate wind, 8.5 mm. rain
95	54	74	Bright and clear, perceptible breeze	15	31/32	17/32	15/32	†	95	53	74	Cloudy A. M., bright and clear P. M.
94	64	79	Bright and clear, high wind	16	18/32	12/32	28/32	†	95	61	78	Bright and clear, moderate breeze
92	64	78	Cloudy, high wind	17	....	....	....	†	100	64	82	Bright and clear, perceptible breeze
88	53	70	Hazy, moderate breeze	18	....	....	....	†	97	63	80	Clear, light breeze
88	50	69	Bright and clear, moderate breeze	19	....	....	....	†	95	68	82	....
93	50	71	Cloudy, moderate breeze	20	....	....	....	†	92	57	74	....
90	62	76	Bright, clear A. M.; cloudy P. M.; high wind	21	....	....	....	†	94	63	79	....
93	63	78	Bright, clear A. M.; cloudy P. M.; trace rain	22	....	....	....	†	94	63	78	....
95	59	77	Bright and clear, high wind	23	108/32	44/32	112/32	†	95	68	82	Bright and clear, light wind
91	50	70	Bright and clear, high wind	24	....	....	....	†	100	65	82	Bright and clear, cloudy toward evening
91	63	77	Cloudy, high wind	25	....	....	....	†	94	64	79	Clear day, light wind
98	57	78	Hazy, moderate breeze	26	24/32	24/32	30/32	†	96	61	79	No. 1 gauge, 3/16 in. rainfall
98	57	77	Hazy, calm	27	....	....	....	†	98	64	81	....
90	57	74	Cloudy, calm	28	....	....	....	†	100	63	81	....
99	54	76	Hazy, calm	29	....	....	....	†	98	70	84	....
98	54	76	Clear, perceptible breeze	30	50/32	40/32	32/32	†	97	62	80	....
				31	13/32	†	†	†	88	62	75	....
89	55	72	....	Total or average.	13.75	9.59	12.56	†	96	62	79	....

able.

‡ Pan submerged.

† No record available.

Date, 1914, June.	Evaporation, in Inches.				Temperature.			Weather conditions.
	Land-pans.		Floating-pans.		Maximum, degrees, Fahr.	Minimum, degrees, Fahr.	Mean, degrees, Fahr.	
	No. 1.	No. 3.	No. 2.	No. 4.				
1	....	†	†	†	95	61	78	
2	....	†	†	†	95	61	78	
3	32/32	†	†	†	101	68	85	No. 1 gauge shows 5/8 in. rainfall.
4	....	....	....	†	99	69	84	
5	....	....	....	†	99	71	85	
6	....	33/32	32/32	†	101	71	86	
7	....	....	....	....	99	66	82	
8	....	....	....	....	100	68	84	
9	....	....	....	....	101	68	85	
10	....	....	....	....	99	64	81	
11	....	....	....	....	98	64	81	
12	....	....	....	....	98	68	83	
13	....	....	....	....	99	68	84	
14	....	....	....	....	100	70	85	
15	....	....	....	....	96	66	81	
16	....	....	....	....	87	66	76	
17	....	....	....	....	89	63	76	
18	....	....	....	....	90	64	77	
19	....	....	....	....	95	66	80	
20	....	....	....	....	93	66	80	
21	....	....	....	....	94	66	80	
22	....	....	....	....	88	64	76	
23	....	....	....	....	89	65	77	
24	....	....	....	....	90	66	78	
25	....	....	....	....	98	66	80	
26	....	....	....	....	96	68	82	
27	....	....	....	....	99	70	85	
28	....	....	....	....	100	66	83	
29	....	....	....	....	101	68	84	
30	....	....	....	....	101	68	85	
Total or average.	1.22	1.03	1.00	†	96	66.5	81	

†No record available.

NOTES.

This table is a record of evaporation and temperature observations, taken after the completion of Table 34; and is a continuation of that table, to which reference is made for descriptions and locations of evaporation pans and other data.

The general scheme of Table 34 (Plate LXVI) is followed in Table 36, except that the El Paso records, considered unsatisfactory, were discontinued in March, 1914; and that observations, here given, covering periods of several days' evaporation, have not been distributed over those days but recorded opposite the date on which the reading was made.

Evaporation records for March 1st to 9th, 1914, taken from Table 34, are given as on that table, *i. e.*, distributed over the days covered, assuming that the daily rate of evaporation for each period was constant.

Evaporation given is net, *i. e.*, rainfall has been accounted for.

evaporation depth (the winter months, as March), the month's evaporation depth at any stated mean monthly temperature may vary greatly in different years. For example, at a mean monthly temperature of 50° Fahr., the monthly evaporation depth at El Paso averages 3.7 in. (see the "straight-line" curve of Fig. 7); but for the different months or years observed, the month's evaporation depth at 50° varies from 2.0 to 4.8 in., or from 54 to 130% of the mean value, and from 100 to 240% of the minimum value.

Also, each of the points plotted on Fig. 7 is for a period of a full month, with all the equalizing of the changing daily evaporation conditions which thus would be effected; while the observations at El Paso in March, 1914, at approximately 51° mean temperature, covered a period of only 16 days, and thus are likely to contain more vagaries and uncertainties and to be less equalized than values derived from observations covering a full month or longer.

Finally, the evaporation depth at El Paso for the full month of March, 1914, averages (as deduced from the 16 days' observations) 2.48 in., or is 33% below the average monthly depth at El Paso at 50°; but it could not be known whether the observed evaporation depth at Lake Conchos for the same month of March varied from its mean in the same or in the opposite direction.

For the reasons given, it was concluded that estimates of yearly evaporation depth based on comparisons of evaporation depths at two widely separated stations observed for short periods only, are unsafe and unreliable, and not worthy of consideration.

Variants B_a and B_b consisted of an expansion (in March, 1914) of the measured pan evaporations at Lake Conchos for the months of January-March (the observations covered only 2 months actual time) into the probable depth of pan evaporation for a complete yearly cycle; and later (in August, 1914), of a similar expansion of the longer-continued pan evaporations of January-June (only 5 months actual time).

The B_a method of such expansion is the division of the observed evaporation depth in pans at Lake Conchos of the 3 months, January-March, by the proportion of the evaporation depth for a full year which is believed to occur during those 3 months (as shown by the evaporation-temperature study of Table 31)—and similarly for the longer-continued pan evaporations of the 6 months, January-June, 1914.

The observed evaporation depths at Lake Conchos from land and floating-pans are summarized in Table 35 for the shorter period (January 13th to March 9th, 1914); and for the longer period (January 13th to June 6th, 1914), they are summarized (from Tables 34 and 36, Plates LXVI and LXVII) in Table 39.

From Table 35, the measured evaporation depths from 3-ft. square floating-pans at Lake Conchos during the shorter period were as given in Table 38, after expanding all the observations of partial months to full months and taking 80% of the evaporation depths from the land-pans as the corresponding depths from floating-pans.

TABLE 38.—EVAPORATIONS FROM 3-FT. SQUARE FLOATING-PANS.

Month.	Pan No. 1 (land-pan). Inches.	Pan No. 2 (floating-pan). Inches.	Pan No. 3 (land-pan). Inches.	Pan No. 4 (floating-pan). Inches.	Average of the four pans. Inches.
January, 1914.....	*6.36	4.96	*5.57	6.82	5.92
February, "	4.25	5.60	3.57	2.94	4.09
March, "	4.40	5.12	5.44	5.74	5.17
Totals and average....	15.01	15.68	14.58	15.50	{ 15.18 / 15.19
Percentage.....	99.0	103.2	96.0	102.1	100

* The evaporation depths given in this table for Pans Nos. 1 and 3 (land-pans) are 80% of the evaporations actually measured in those land-pans.

From Table 31, the proportions of the total yearly evaporation depths at Lake Conchos which occur in the 3 months, January, February, and March, are as follows:

Depth in January..... 4.8% of total depth in year.
" " February ..... 5.6 " " " " "
" " March ..... 7.4 " " " " "
Depth for the 3 months.....17.8% " " " " "

Hence, if 15.18 in. is 17.8% of the yearly evaporation depth, the total yearly depth must be $(15.18 \div 0.178) = 85.3$ in.; or the total yearly evaporation depth at Lake Conchos from 3-ft. square floating-pans is (by Method B_a and for the shorter period) 85.3 in.

By Subsidiary Conclusion (*c*), the evaporation depth from a large reservoir is 62% of that from a 3-ft. square pan floating thereon; hence, by Method B_a and for the shorter period, the yearly evaporation depth from Lake Conchos is $(85.3 \times 0.62) = 52.9$ in.

The yearly evaporation depth by Method B_a for the longer period is obtained from Table 39.

From Table 39, the measured evaporation depths from the four 3-ft. square pans were as given in Table 40 for the longer period, after expanding all observations for parts of months to full months and modifying the depths measured in the land-pans to their corresponding depths (80%) in floating-pans.

TABLE 39.—SUMMARY OF PAN-EVAPORATION MEASUREMENTS MADE AT LA BOQUILLA (LAKE CONCHOS) IN 1914.

EVAPORATION PAN.	PAN NO. 1 (LAND-PAN).						PAN NO. 2 (FLOATING-PAN).					
Month (of 1914).	Jan.	Feb.	Mar.	Apr.	May.	June.	Jan.	Feb.	Mar.	Apr.	May.	June.
Number of days measured	19	27	31	30	31	3	9	21	22	27	30	3
Total depth measured, in inches	4.87	5.1	6.28	10.38	13.75	1.22	1.44	4.2	3.60	8 72	12.56	1.00
Average daily evaporation depth, in inches	0.26	0.19	0.20	0.35	0.44	0.41	0.16	0.20	0.16	0.32	0.42	0.33
Corresponding depth for full month, in inches	7.95	5.30	6.28	10.38	13.75	12.20	4.96	5.60	5.07	9.70	12.98	10.00
Depth for 3-ft. floating-pan, for month, in inches	6.36	4.25	5.02	8.30	11.00	9.76	4.96	5.60	5.07	9.70	12.98	10.00
Mean temperature, in degrees, Fahrenheit, for period of measurement.	64	56	58	72	79	80	64.5	56	58	73	79	85
Mean temperature, in degrees, Fahrenheit, for entire month	62	56	58	72	79	81	62	56	58	72	79	81

EVAPORATION PAN.	PAN NO. 3 (LAND-PAN).						PAN NO. 4 (FLOATING-PAN).					
Month (of 1914).	Jan.	Feb.	Mar.	Apr.	May.	June.	Jan.	Feb.	Mar.	Apr.	May.	June.
Number of days measured	18	22	31	30	28	3	2	6	2	.....	.....	.....
Total depth measured, in inches	4.04	3.5	6.41	9.63	9.59	1.03	0.44	0.63	0.37	.....	.....	.....
Average daily evaporation depth, in inches	0.22	0.16	0.21	0.32	0.34	0.34	0.22	0.10	0.19	.....	.....	.....
Corresponding depth for full month, in inches	6.96	4.46	6.41	9.63	10.62	10.30	6.82	2.94	5.74	.....	.....	.....
Depth for 3-ft. floating-pan, for month, in inches	5.57	3.57	5.13	7.70	8.50	8.24	6.82	2.94	5.74	.....	.....	.....
Mean temperature, in degrees, Fahrenheit, for period of measurement.	64	56	58	72	79	85	60	58	57	.....	.....	.....
Mean temperature, in degrees, Fahrenheit, for entire month	62	56	58	72	79	81	62	56	58	.....	.....	.....

TABLE 40.—EVAPORATIONS FROM 3-FT. SQUARE FLOATING-PANS.

Month.	Pan No. 1 (land-pan). Inches.	Pan No. 2 (floating-pan). Inches.	Pan No. 3 (land-pan). Inches.	Pan No. 4 (floating-pan). Inches.	Average of the four pans. Inches.
January	6.36	4.96	5.57	6.82	5.92
February	4.25	5.60	3.57	2.94	4.09
March	5.02	5.07	5.13	5.74	5.24
April	8.30	9.70	7.70	*....	8.57
May	11.00	12.98	8.50	....	10.82
June	9.76	10.00	8.24	....	9.33
Totals and averages...	44.69	48.31	38.71	....	{ 43.97
Percentages	101.9	110.0	88.2	....	{ 43.90 100

* Observations in Pan No. 4 were abandoned March 9th, because of too frequent submergences of this pan by waves.

From Table 31, the proportions of the total yearly evaporation depth at Lake Conchos occurring in each of the 6 months, January-June, are as follows:

Depth in January..... 4.8% of total depth in year.
" " February ..... 5.6% " " " " "
" " March ..... 7.4% " " " " "
" " April ..... 9.4% " " " " "
" " May .....11.5% " " " " "
" " June .....11.7% " " " " "
Total for the 6 months..50.4% of total depth in year.

Hence, on the assumption that 50.4% of the yearly evaporation depth equals 43.97 in., the total yearly evaporation depth at Lake Conchos must be $(43.97 \div 0.504) = 87.2$ in. from 3-ft. square floating-pans and—by Method B_a and for the longer period—$(87.2 \times 0.62) = 54.1$ in. from the surface of the lake itself.

The expansion by Method B_b from the 3 and 6 months' observed evaporations to that for a full year is made by evaporation-temperature relations, as follows:

By Method A, the yearly evaporation depth from 3-ft. square pans floating on Lake Conchos is 84.74 in. (see Table 31); and (also from Table 31) the evaporation depths for the 3 months, January-March, and the 6 months, January-June, should be, respectively $(4.09 + 4.75 + 6.23) = 15.07$ and $(4.09 + 4.75 + 6.23 + 7.95 + 9.72 + 9.90) = 42.64$ in.

From Tables 38 and 40, the average pan evaporations (from 3-ft. square floating-pans) actually observed at Lake Conchos were as follows:

3 months, January-March.....15.18 in.
6 " January-June.....43.97 "

From Tables 35 and 39 and Fig. 15, the observed mean monthly temperatures at Lake Conchos and their corresponding theoretical monthly evaporation depths were as given in Table 41.

TABLE 41.—THEORETICAL EVAPORATION DEPTHS FROM 3-FT. SQUARE FLOATING-PANS, CORRESPONDING TO OBSERVED MEAN MONTHLY TEMPERATURES.

Month.	JANUARY-MARCH, 1914.		JANUARY-JUNE, 1914.	
	Observed mean monthly temperature, in degrees, Fahrenheit.	Corresponding theoretical monthly evaporation depth, in inches.	Observed mean monthly temperature, in degrees, Fahrenheit.	Corresponding theoretical monthly evaporation depth, in inches.
January.....	62	6.03	62	6.03
February.....	56	4.76	56	4.76
March.....	55	4.55	58	5.20
April.....	..	....	72	8.16
May.....	..	....	79	9.64
June.....	..	....	81	10.05
Totals.....	....	15.34	....	43.83

Hence the comparison of the observed evaporation depths at Lake Conchos with the theoretical depths deduced by evaporation-temperature relations are as given in Table 42.

TABLE 42.—COMPARISON OF OBSERVED AND THEORETICAL EVAPORATION DEPTHS.

Item.	3 months—January-March.	6 months—January-June.
Observed evaporation depths....	15.18 in. = 100%	43.97 in. = 100%
Theoretical evaporation depths :		
From temperatures of 1912-13.	15.07 " = 99.3%	42.64 " = 97.1%
" " " 1914....	15.34 " = 101.1%	43.84 " = 99.8%
Mean of two.....	100.2%	98.45%

Thus, from the results of the three (2) months' pan observations at Lake Conchos, the theoretical evaporation depths there (those estimated by Method *A*) apparently are 0.2% too great; and from the results of the six (5) months' observations, 1.55% too small. The evaporation depths at Lake Conchos, as estimated by Method B_b, therefore, are as given in Table 43.

The estimation of the evaporation depth by Method *C* will now be considered.

Method C.—This method consists in estimating the inflow to the lake throughout two practically rainless periods, deducting therefrom

the measured outflow during such periods, and then, from the observed elevations and known areas of the lake surface, estimating the depth on the lake surface corresponding to the excess of inflow over outflow, which excess must have been (at least to a close approximation) the loss by evaporation.

TABLE 43.—YEARLY EVAPORATION DEPTHS AT LAKE CONCHOS, BY METHOD B_b.

From three (2) months' pan observations.	From six (5) months' pan observations.
In 3-ft. square floating-pans: *84.74 (= A) ÷ 1.002 = 84.5 in. = 98.2%	In 3-ft. square floating-pans: 84.74 (= A) ÷ 0.9845 = 86.2 in. = 100%
In Lake Conchos: *52.5 (= A) ÷ 1.002 = 52.4 in. = 98.2%	In Lake Conchos: 52.5 (= A) ÷ 0.9845 = 53.4 in. = 100%

*From Table 33.

The two periods thus used are the 4 months, October, 1913-January, 1914, and the 7 months, October, 1913-April, 1914. Since January, 1912, continuous gaugings have been made of the flow of the Rio Conchos at Pilar de Conchos, 40 miles up stream from the La Boquilla Dam and 2 miles above the upper end of the filled reservoir. There are a number of arroyos between Pilar de Conchos and the dam, however, which discharge much water into the lake during rainy and flood seasons; and it is only during rainless periods that the Rio Conchos discharge, as measured at Pilar de Conchos, constitutes the total water supply to the reservoir. The 7 months, October, 1913-April, 1914, were such a practically rainless period.

The inflow and outflow of Lake Conchos during the 4 and 7-month periods are estimated from the continuous stream gaugings, at Pilar de Conchos and just below the La Boquilla Dam, to have been as given in Table 44.

TABLE 44.—EXCESS OF INFLOW TO LAKE CONCHOS OVER OUTFLOW.

Item.	Millions of Cubic Meters.	
	4 months, October-January.	7 months, October-April.
Inflow as gauged at Pilar de Conchos	44.193	59.184
Outflow as gauged just below La Boquilla Dam.	*20.095	†36.963
Gross volume of water reservoired	24.098	22.221

* These volumes include, not only the stream flows as measured just below the power-house, but also the measured seepage losses entering the stream below that gauging station, plus 10% of such measured seepage losses added to cover known but unmeasurable additional seepage losses.

† Due to fuller and more complete field reports at the later date, the seepage losses of October–January are larger and more exact and complete in the volumes of outflow in Table 44 for October–April than in the volumes for October–January only given in the table separately.

The fluctuations of the lake surface during the two periods are given in Table 45.

TABLE 45.—Elevations and Rises of Surface of Lake Conchos, etc.

Item.	4 months, October-January.	7 months, October-April.
	Meters.	Meters.
Elevation of surface of lake above sea, at beginning and end of period	1 288.96 and 1 289.28	1 288.96 and 1 288.92
Mean elevation of lake surface during the period	1 289.12	1 288.94
Rise of lake surface during the period	+ 0.320	— 0.04
Rainfall during period (mean of the two rain-gauges at La Boquilla and Pilar de Conchos) to be subtracted	— 0.058	— 0.077
Rise of lake surface after eliminating the effect of rain falling directly on lake; or rise corresponding to net reservoired water [(inflow less outflow), less depth lost by evaporation]	+ 0.262	— 0.117
	Millions of cubic meters.	Millions of cubic meters.
Volume of reservoir storage per meter of rise, at mean surface elevation	*42.4	*42.0
Gross volume of water reservoired during period, including that lost afterward by evaporation, but excluding that gained by direct rainfall (from Table 44)	24.098	22.221
	Meters.	Meters.
Rises of lake surface (at mean elevations of 1 289.12 and 1 288.94 m.) corresponding to above 24.098 and 22.221 millions of cubic meters; or rises which would have occurred if there had been no evaporation loss from lake surface	† + 0.568	† + 0.529
Rises which actually did occur after losses by evaporation and correction on account of rainfall (from above)	+ 0.262	— 0.117
Differences in theoretical and actual rises above; or estimated evaporation depths from the lake in the 4 months, October-January, and the 7 months, October-April [0.568–0.262 m. and 0.529 — (— 0.117) m.]	0.306	0.646
Estimated evaporation depths from the lake during the 4 months, October-January, and the 7 months, October-April, in inches (3.28 × 12 × 0.306 and 3.28 × 12 × 0.646)	12.04 in.	25.40 in.

* From curves and tables of the reservoir areas and capacities.
† (24.098 ÷ 42.4) = 0.568, and (22.221 ÷ 42.0) = 0.529.

During May, 1914, the rainy season began, and many local flood flows into the lake occurred from arroyos between Pilar de Conchos and La Boquilla. Hence the measurement of the inflow to the lake and the use of Method *C* were impracticable after the end of April, 1914.

The estimated evaporations, of 12.04 in. in the 4 months and 25.40 in. in the 7 months, were expanded into the evaporation depths for a full year by two variants, C_a and C_b.

Method C_a is identical with that used in Method B_a, and consists in expanding by making use of the proportion of the total yearly evaporation depth which occurs in the 4 and 7 months, as in Table 46 (from Table 31).

TABLE 46.—THE PROPORTIONS OF THE TOTAL YEARLY EVAPORATION DEPTH OCCURRING DURING THE 4 MONTHS, OCTOBER-JANUARY, AND THE 7 MONTHS, OCTOBER-APRIL.

Month.	4 months, October-January.	7 months, October-April.
October	7.7%	7.7%
November	5.1	5.1
December	4.1	4.1
January	4.8	4.8
February	...	5.6
March	...	7.4
April	...	9.4
Totals	21.7%	44.1%

Hence, if 12.04 in. = 21.7%, and 25.40 in. = 44.1%, of the total yearly evaporation depth from the surface of Lake Conchos, the total yearly evaporation depth, as estimated by Method C_a, must be:

From the consideration of the 4 months,
October-January $(12.04 \div 0.217) = 55.4$ in.
From the consideration of the 7 months,
October-April $(25.40 \div 0.441) = 57.7$ in.

Method of expansion C_b is identical with that used in B_b, and consists in expanding from the 4 and 7 months to the full year by making use of evaporation-temperature relations, as follows:

By Method A, the yearly evaporation depth from the surface of Lake Conchos is 52.5 in. (see Table 33).

From Tables 37 and 39, and Fig. 15, the observed mean monthly temperatures at Lake Conchos and their corresponding theoretical monthly evaporation depths were as given in Table 47.

By the relation adopted, 62%, the theoretical evaporation depths in Table 47 from 3-ft. square floating-pans would correspond to the following evaporation depths from the surface of Lake Conchos:

From the consideration of the 4 months,
October-January $(18.18 \times 0.62) = 11.25$ in.
From the consideration of the 7 months,
October-April $(36.30 \times 0.62) = 22.50$ in.

TABLE 47.—THEORETICAL EVAPORATION DEPTHS FROM 3-FT. SQUARE FLOATING-PANS, CORRESPONDING TO OBSERVED MEAN MONTHLY TEMPERATURES.

Month.	4 MONTHS, OCTOBER–JANUARY.		7 MONTHS, OCTOBER–APRIL.	
	Observed mean monthly temperature, in degrees, Fahrenheit.	Corresponding theoretical monthly evaporation, in inches.	Observed mean monthly temperature, in degrees, Fahrenheit.	Corresponding theoretical monthly evaporation, in inches.
October.....	60	5.60	60	5.60
November..	52	3.90	52	3.90
December...	46	2.65	46	2.65
January....	62	6.03	62	6.03
February...	..	....	56	4.76
March.....	..	....	58	5.20
April.....	..	....	72	8.16
Total.....	..	18.18	..	36.30

The above theoretical evaporations from the surface of the lake, estimated by Method *A*, are, respectively (11.25 ÷ 12.04*) = 93.5% and (22.50 ÷ 25.40*) = 88.6% of those estimated by Method *C*; and, assuming the same relations to hold for the entire year and to apply to the 52.5 in. yearly evaporation depth estimated by Method *A*, the estimated yearly evaporation depth from the surface of the lake is as follows by Method C_b:

From the consideration of the 4 months,
October-January (52.5 ÷ 0.935) = 56.2 in.
From the consideration of the 7 months,
October-April (52.5 ÷ 0.886) = 59.3 in.

For various reasons mentioned on pages 1845 to 1848, it is believed that nearly all the uncertainties incident to the estimation of the yearly evaporation depth from Lake Conchos by Method *C* are of a nature tending to make the depths resulting from that method somewhat greater than the actual; or that the true yearly evaporation depth in all probability is somewhat less than the depths of 55.4 and 57.7 in. and 56.2 and 59.3 in. resulting from Methods C_a and C_b, respectively.

In Tables 3 and 4 are given a summary and a percentage comparison of the five values of yearly evaporation depth from Lake Conchos estimated (for the longer period of observation) by Methods *A*, B_a, B_b, C_a, and C_b. In Table 5 is given a percentage-comparison of the five values, as estimated from both the longer and the shorter periods. On page 1848 is given the value of 55 in. (1.40 m.) adopted in March,

* From Table 45.

1914 (and confirmed in August), as the safe gross yearly evaporation depth from the surface of the lake, and the reasons for that adoption. And in Table 6 is given the division of the 55 in. yearly depth into the depths of the 12 months, also the net monthly evaporation depths after reducing the gross depths by the average depth of rain each month falling directly on the reservoir surface.

This completes the Appendix and this study of evaporation—except that data now are available to check, from the Lake Conchos investigation itself, the value of 62% which was adopted as Subsidiary Conclusion (*c*).

(*c*). Evaporation Depth from a Large Reservoir, as a Percentage of That from a 3-ft. Square Pan Floating Thereon. (As Checked from Lake Conchos Data.)

The yearly evaporation depth from Lake Conchos itself has been estimated from 4 months' and 7 months' observations by Methods C_a and C_b. The yearly evaporation depths from 3-ft. square pans floating on the lake have been estimated by Method A, and from 2 months' and 5 months' observations by Methods B_a and B_b. All these yearly depths of evaporation are summarized in Tables 48 and 49.

TABLE 48.—YEARLY DEPTH OF EVAPORATION FROM THE SURFACE OF LAKE CONCHOS.

Method of estimation.	FROM LAKE OBSERVATIONS OF:	
	4 months.	7 months.
C_a — from page 1898.....	55.4 in. = 99.3%	57.7 in. = 98.8%
C_b — from page 1899.....	56.2 " = 100.7%	59.3 " = 101.2%
Mean of C_a and C_b	55.8 in. = 100%	58.5 in. = 100%

TABLE 49.—YEARLY DEPTH OF EVAPORATION FROM 3-FT. SQUARE PANS FLOATING ON LAKE CONCHOS.

Method of estimation.	FROM PAN OBSERVATIONS OF:	
	2 months.	5 months.
*A—from Table 32.....	*84.7 in. = 99.8%	*84.7 in. = 97.7%
B_a— " pages 1892-1894....	85.3 " = 100.5%	87.2 " = 100.6%
B_b— " Table 43.....	84.5 " = 99.5%	86.2 " = 99.4%
Mean of A, B_a, and B_b.....	84.85 in. = 100%	86.03 in. = 99.2%
Mean of B_a and B_b	84.9 " = 100%	86.7 " = 100%

* Method A includes no pan observations at Lake Conchos, but temperature observations only.

Various combinations of Tables 48 and 49, giving various values of c, are given in Table 50:

TABLE 50.—VARIOUS VALUES OF c FROM LAKE CONCHOS DATA.

Combinations of lake evaporation and pan evaporation.	From Observations of:	
	4 and 2 months.	7 and 5 months.
$C_a \div B_a$	(55.4 ÷ 85.3) =65.0%	(57.7 ÷ 87.2) = 66.2%
$C_b \div B_b$	(56.2 ÷ 84.5) =66.5	(59.3 ÷ 86.2) = 68.8
*Mean of C_a and C_b ÷ mean of B_a and B_b	(55.8 ÷ 84.9) =65.8	(58.5 ÷ 86.7) =*67.5
Mean of C_a and C_b ÷ A	(55.8 ÷ 84.7) =65.9	(58.5 ÷ 84.7) = 69.1
Mean of C_a and C_b ÷ mean of A, B_a and B_b	(55.8 ÷ 84.85)=65.8	(58.5 ÷ 86.03) = 68.0
Mean of all above values	65.8%	67.9%

* Believed to be the most accurate and reliable value.

The values of c in Table 50 vary through only a small range, the difference between the highest and lowest values (69.1 and 65.0%) being only 4.1 per cent.

The 67.5% is felt to be the most accurate and reliable value of all —because of the longer periods of observation, because the lake and the pan observations of Methods *C* and *B* cover nearly the same periods of time, and because Method *B* makes use of the local pan evaporations at Lake Conchos and Method *A* does not.

The 67.5% value of c corresponds to yearly evaporations of 58.5 in. from the surface of the lake and 86.7 in. from 3-ft. square pans floating thereon. As has been stated already, it is known that the evaporation depths estimated by Method *C* err in being too great, and that, to secure accurate values, they must be decreased by the depths of water absorbed by the bottom of the reservoir during the period of observation (which was its first time of filling).

The present operation is a check on the correctness of the value 62% for c. Assuming 62% to be the correct value, then the yearly evaporation depth from the surface of Lake Conchos would be (86.7 in. pan evaporation $\times$ 0.62) = 53.75 in. This is only (58.5 — 53.75) = 4.75 in. less depth than the most reliable value of lake evaporation as estimated by Method *C*. It is shown in the discussion of Table 4, however, that there is no improbability and even much probability of the absorption, etc., even of 5.05 in. depth of the reservoired water during the period of observation.

Hence, from the check of the value of c by the Lake Conchos observations, it may be stated that those observations show the evaporation depth from large reservoirs to be certainly less than 67.5% (and in all probability as little as 62%) of that from 3-ft. square pans floating thereon.

DISCUSSION

Mr. Mathewson.

T. K. MATHEWSON,* M. AM. SOC. C. E. (by letter).—In December, 1911, the writer investigated a proposed hydro-electric power project on the lower reaches of the Conchos River, and was confronted with the almost complete absence of meteorological data in Chihuahua noted by the authors. He also visited the site of the Conchos Dam at that time. This work was in its early stages, and was being carried on by the contractors, S. Pearson and Son.

In the course of the writer's work for the Guanajuato Power and Electric Company, the depth of yearly evaporation in large reservoirs, decided on and used in a long series of storage investigations in the State of Michoacán, was 1.45 m. (4.75 ft.). This agrees closely with the 1.40 m. (4.59 ft.) determined and used at Lake Conchos. The elevation above sea level of that part of Michoacán is about 2 000 m. (6 560 ft.). The average yearly rainfall from 1892 to 1913, inclusive, was 705.6 mm. (27.78 in.).

Michoacán has the best meteorological records of any of the Mexican States in which the writer has worked. A meteorological observatory was established at Morelia, the State capital, in May, 1908, in charge of Mr. Reyes, a young man of great ability and enterprise. At each county seat there is a well-equipped station, with a volunteer observer to record the rainfall, temperature, and wind. The writer prizes a gift of the complete files of the monthly bulletins issued by the observatory, which he has had bound into a volume. In May, 1912, the whole service was stopped by the Revolution, but data of great value were collected and tabulated during the four years of its existence.

Not far from Morelia is a hydro-electric power plant of the Guanajuato Company, where records of rainfall and temperature are kept. Besides being the State capital, Morelia is a cathedral city, and the records of the church meteorological observatory go back to January, 1892. By combining the records of the Church, the State, and the Power Company, there is a continuous record of rainfall, by months, in this part of Michoacán for more than 23 years.

The evaporimeter at Morelia is a land-pan. It is not protected from sunshine or wind, but the bottom and sides are protected by the earth and sod. In order to make practical use of the record of this evaporimeter, it was desired to know what proportion of the annual depth of evaporation shown by it might be expected in a large body of water. Very little information could be found on the subject, although several investigators gave values of from one-half to three-fourths.

* Muscatine, Iowa.

Mr. Mathewson.

A farm reservoir was found, having neither inflow nor outflow during a long period of time in the dry season, and a record of the water level was kept, proper correction being made for the only rain storm which occurred. The soil was impervious, so that the fall in the water level could be due to nothing but evaporation. The long record of the evaporimeter was compared with the relatively short record at the reservoir, and a relationship established, which the writer remembers as 56%, instead of the 62% found at Lake Conchos. At any rate, an annual depth of 1.45 m. (4.75 ft.) was determined for the reservoir, as compared with a depth of nearly 3 m. (9.84 ft.) by the evaporimeter. These figures are from memory and a few scanty notes, as the writer was unable to bring his books and papers with him on leaving Mexico, and they are not now accessible.

The thorough and extensive study made by the authors at Lake Conchos is a real contribution to knowledge in a field that has not been investigated very much until recently. The Mexican Northern Power Company may well be congratulated that it has been able to carry its great work to completion during such troublous times.

*The writer is sorry to have fallen into an error through trusting too much to his memory. Reports on file at the home office of the Guanajuato Power Company in the United States show that the percentage determined and used in Michoacán was 52%, and not 56% as stated. Taking 52% of the annual evaporation of 1909, a very dry year, gives a depth of evaporation of 1 717.8 mm., or approximately 1.72 m. (5.64 ft.), instead of 1.45 m. (4.75 ft.) as before stated. This corresponds to an annual evaporation of 3.303 m. (10.83 ft.) in the evaporimeter. Difference in latitude, in temperature, and in elevation above sea level may account for the greater evaporation in Michoacán, as compared with that at Lake Conchos. The writer regrets that the temperature records at Morelia were not placed on file in the United States, where they would now be accessible for making comparisons. All the other figures in the writer's previous discussion are correct as stated.

Mr. Chandler.

E. F. CHANDLER,† ASSOC. M. AM. SOC. C. E. (by letter).—The conclusions drawn by the authors from this interesting and comprehensive investigation seem, in general, to be well founded and safely applicable in the region studied. The writer, however, desires to emphasize one point mentioned only briefly, and to call attention to the impropriety, and indeed danger, of applying the numerical results embodied in its conclusions to any distant region of different topography and climate.

Rainfall varies much from year to year, so that the maximum annual rainfall is three times the minimum annual rainfall, more

* Mr. Mathewson has submitted this paragraph in order to correct the foregoing part of his discussion.

† University, N. Dak.

Mr. Chandler.

or less, depending on the climatic conditions of the region. The variation in the total annual run-off of any stream may be expected to be very much greater, comparatively, even than that in the annual rainfall of its drainage area. The variations in mean annual temperature and in total annual evaporation at any point, however, are comparatively small, and if an approximately correct relation between the temperature and the evaporation can be discovered, it will be of much use for preliminary estimates where records of evaporation are lacking. Fortunately, long-extended temperature records exist at very many more points and at points more widely distributed than evaporation records.

Since April, 1905, the writer has supervised the continuous maintenance, through the open season of each year, of an evaporation record at University, N. Dak. Daily observations begin soon after the spring break-up of the streams, and continue until they freeze in the fall. There have been only a few very short interruptions, so that the total record covers more than 67 months in the 10 years from 1905 to 1914, inclusive. The latitude of the station is 48°, its longitude 97°, and its altitude 820 ft. above sea level. The evaporation is from a floating-pan, 3 ft. square and 18 in. deep, of the standard U. S. Geological Survey pattern, in a narrow pool or reservoir having an area of about 2 acres and a maximum depth of 10 ft., formed by damming a small stream flowing through a shallow depression in the level Dakota prairie. Conditions are such that the temperature of the water in the pan is not ordinarily found to vary appreciably from that in the surrounding reservoir.

The monthly mean temperatures for each month, from April to November, since records were begun at this station, have varied (for those months when records for the entire month were obtained) from 39 to 73° Fahr., and the evaporation for the month from 1.29 to 7.09 in. The mean evaporation for the period of record for the 10 years, averaging 205 days (from April 14th to November 6th), has been 27.05 in. The mean evaporation for the 6 months (from April 1st to October 31st) has been 25.2 in., and the mean temperature for that period 60° Fahr.

If a table is made showing the mean temperatures and evaporations for the 59 complete months included in the period and for the 12 incomplete months in which at least 10 days' record was secured (these incomplete months being filled out by proportion), and if from this table of 71 values for this station, a diagram (Fig. 16) is prepared according to the method of the authors, it is found to be satisfied fairly well from temperatures 35 to 75° Fahr. by the "straight-line" curve representing the equation,

$$E = 0.135\ T - 4.00,$$

in which E is the evaporation, in inches, during the month, and T the mean temperature of the month, in degrees, Fahrenheit. Mr. Chandler.

(The curve ought properly to depart somewhat from a straight line for low temperatures, perhaps below 40°, because there is always some evaporation, even at the lowest temperatures, 50 or 60° below the freezing point; but as the evaporation then is small and is measured with difficulty, it has not been determined for these records, and consideration of it here will be omitted.)

It is evident that this well-founded record, maintained according

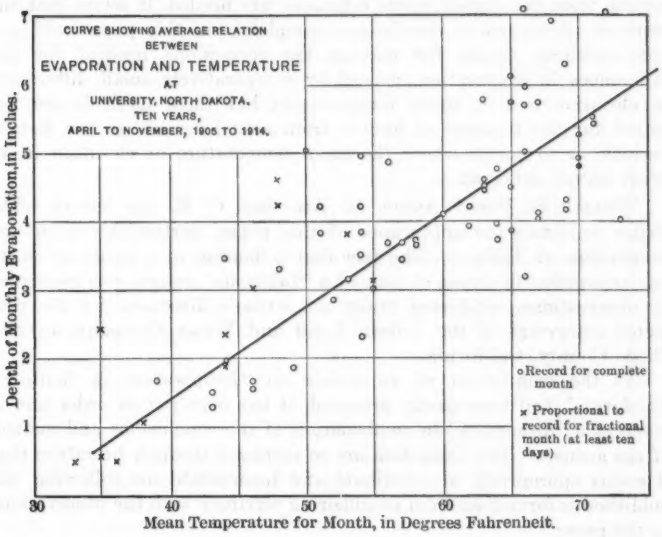


FIG. 16.

to the standard methods, does not in the slightest conform to the numerical conclusions drawn for the Great Plateau in Texas and New Mexico. If the conclusions deduced by the authors for that region were applicable in the Red River Valley of North Dakota, we might use or extend the authors' diagram (*a*), Fig. 4, and Fig. 13, from which, instead of the equation previously stated, we could deduce, approximately,

$$E = 0.20\ T - 10.00,$$

and then, by computing for each month here, from its mean temperature, the expected evaporation, and adding, we should obtain for the 6 months, from April 1st to October 31st, a total computed evaporation of only about 11 in., instead of the 25 in. that actually occurs.

Mr. Chandler.

This disagreement is so great as to corroborate strikingly the statement of the authors immediately after Tables 23 and 24, that apparently the tables are not applicable with safety to places outside of the Great Plateau.

Any attempt to use directly and without change the tables presented in this paper for the purpose of estimating evaporation in other distant portions of the Continent would be likely to lead to incorrect and inexcusably absurd results. If good records of evaporation are available, extending through a period of several years, at some point not remote from the region where estimates are needed, it seems that the methods illustrated so clearly and completely in this paper will provide excellent means for making the corrections needful for the differences in evaporation, caused by comparatively small differences in elevation and in mean temperature; but these methods are not suited for the transfer of figures from actual records to far distant regions, or to points where the mean temperature or elevation is, in great degree, different.

Mr. Post.

WILLIAM S. POST,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer welcomes the appearance of this paper, particularly as to the conclusions of Table 2. The fact that a floating or a land-pan shows an evaporation in excess of that of a "full-scale" reservoir is confirmed by observations, conducted under the writer's direction, for the projected reservoirs of the Volcan Land and Water Company, in San Diego County, California.

As the manuscript of an article on "Evaporation in Southern California" had been partly prepared, it has been put in order and is given as a "discussion" in confirmation of the conclusions and method of the authors. The basic data are so scattered through literature that it seems appropriate to contribute and incorporate the following unpublished information from neighboring territory with the observations in the paper.

The results of the writer's study in Southern California, put in the form of Table 2, as tentative conclusions, are as follows:

2 (*b*)—In Southern California the total annual evaporation from a floating-pan is apparently very nearly equal to that from a land-pan, but it shows decided variations, month by month, during the season for the ratio, $\frac{\text{floating-pan}}{\text{land-pan}}$, somewhat as follows:

December, January, and February	about	150	per cent.
March, April, May, and June	"	108	" "
July, August, September, October, and November	"	90	" "
Mean (33 months compared, of which 5 are for a pair of pans close together, and			

* San Diego, Cal.

the rest for pans 2 miles apart, and not under precisely identical conditions)....about 99 per cent. Mean of foregoing pair of pans close together, 5 months (May to September, inclusive).. " 98 " "

Mr. Post.

2 (*c*)—Evaporation depth from a large reservoir equals about 81% of that from a 3-ft. floating-pan.

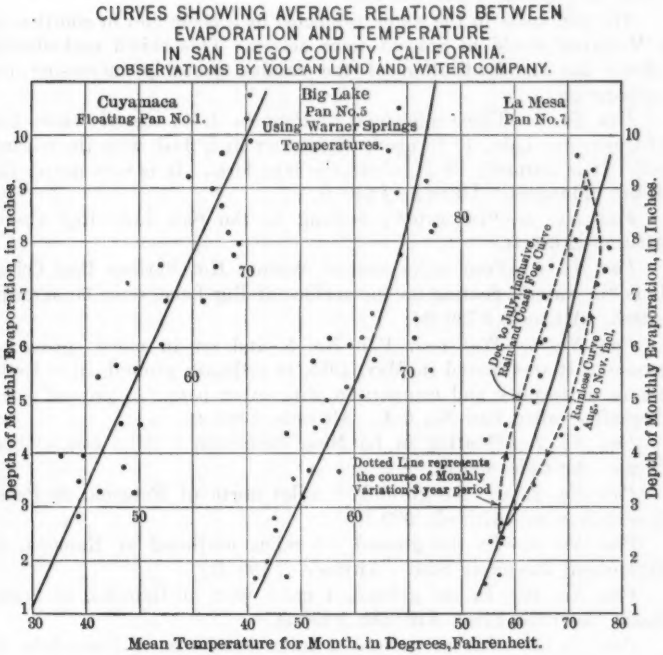


FIG. 17.

DETAIL OF STUDY OF EVAPORATION IN SOUTHERN CALIFORNIA.

The Volcan Land and Water Company and the Cuyamaca Water Company have maintained eleven pans in various locations in San Diego County, which were placed and observed under the writer's direction. These pans are all a standard Weather Bureau type, 3 ft. square and 1½ ft. deep, except certain temporary ones which are 9 in. square. Rain gauges are placed near each pan.

Mr. Post.

Description of Pans.

Pan No. 1.—West margin of Cuyamaca Lake, near the divide between the Pacific Ocean and the Salton Basin, about 40 miles from the Coast. Altitude, 4 620 ft. In the winter the prevailing winds, rain-bearing, are from the southwest, clearing from the north; in summer, desert conditions exist, varying from summer thunder storms of the Arizona type in July and August to extremely dry southeast winds in September and October.

The pan floats in the water, protected by a boom and in addition by a V-shaped shield to prevent wave action. This shield undoubtedly affects the standardization of the results, probably increasing the evaporation.

Pan No. 2.—Three miles east of Pan No. 1, on easterly shore line of Cuyamaca Lake, 10 ft. above high-water line, flush with the natural soil. It is normally 20 ft. above the lake level. It is also nearer the desert influences. Altitude, 4 640 ft.

Pan No. 4.—"In water", floating in the San Luis Rey Creek. Altitude, 2 620 ft.

Pan No. 5.—Four miles west of Warner Hot Springs Post Office. It is "in water", floating on the surface of Big Lake, some 50 acres in extent. Altitude, 2 780 ft.

Pan No. 6.—Was near Pan No. 5, and set in moist spring-fed ground. It was moved in May, 1915, to ordinary ground, in order to secure uniformity and comparison with other pans "in ground", and thereafter called Pan No. 6-*A*. Altitude, 2 800 ft.

Pan No. 7.—Floating in La Mesa Reservoir, 5 miles east of San Diego. Altitude, 480 ft.

Pan No. 8.—In the ground, 4 miles north of Ramona, at Pamo Reservoir Site. Altitude, 900 ft.

Pan No. 9.—In the ground, 10 miles northeast of Ramona, at Sutherland Reservoir Site. Altitude, 2 100 ft.

Pan No. 10.—In the ground, 4 miles west of Ramona, at Santa Maria Reservoir Site. Altitude, 1 400 ft.

Pan No. 11.—In the ground, 5 miles southwest of Escondido, at Carroll Reservoir Site. Altitude, 230 ft.

Pans Nos. 8, 9, and 10 were 9 in. square until December, 1914, when standard 3-ft. square pans were put in.

Data have also been published in *Bulletin No. 81,* United States Geological Survey, Water Supply Papers, for the Sweetwater Reservoir; and the elaborate observations of C. H. Lee, Assoc. M. Am. Soc. C. E., in *Bulletin No. 294,* U. S. Geol. Survey, Water Supply Papers, showing the determinations of the Los Angeles Aqueduct near Independence, Cal.

Mr. Post.

TABLE 51.—GROSS EVAPORATION RECORDS. FLOATING 3-FT. SQUARE PANS. VOLCAN LAND AND WATER COMPANY.

ELEVATION.	4 620 FT.		2 620 FT.		2 780 FT.		480 FT.	
	Pan No. 1.		Pan No. 4.		Pan No. 5.		Pan No. 7.	
Month.	Evaporation, in inches.	Mean temperature.	Evaporation, in inches.	Mean temperature.	Evaporation, in inches.	Mean temperature.*	Evaporation, in inches.	Mean temperature.
January, 1913...	.....	34.8	(3.00)	39.8	(2.00)	42.9	1.59	54.1
February, 1913..	.....	37.5	4.12	44.8	3.30	44.4	4.23	54.5
March, 1913.....	.....	39.2	3.79	47.8	4.31	47.5	4.51	58.4
April, 1913.....	.....	48.6	4.17	50.8	5.71	52.6	6.19	64.3
May, 1913.....	.....	56.1	3.89	55.7	6.03	59.3	7.54	66.9
June, 1913.....	9.25	59.0	4.41	60.0	5.76	63.9	6.98	65.7
July, 1913.....	8.18	66.6	4.42	70.2	6.51	71.6	9.64	71.7
August, 1913....	7.94	68.2	3.72	72.0	5.53	74.2	8.45	76.8
September, 1913.	9.68	65.4	4.01	68.2	6.18	71.3	8.21	75.4
October, 1913....	6.86	54.8	4.93	59 ±	6.29	61.7	6.46	70.2
November, 1913..	5.63	45.1	3.30	49.5	3.67	51.7	3.42	63.8
December, 1913..	2.85	38.5	3.40	42.2	2.79	45.0	2.70	57.3
	.....	51.2	47.16	55.0	58.08	57.2	69.92	64.9
January, 1914...	3.28	42.4	2.76	46.0	1.69	47.7	2.37	57.6
February, 1914..	5.46	42.0	(3.20)	47.5	3.50	49.0	2.94	60.1
March, 1914.....	7.20	47.4	4.11	51.2	4.60	54.2	6.15	65.2
April, 1914.....	5.60	49.4	4.24	54.1	4.98	55.2	6.16	65.7
May, 1914.....	7.53	53.9	4.87	57.0	5.23	58.9	5.48	64.9
June, 1914.....	8.99	63.4	6.31	64.3	6.61	63.7	7.76	70.4
July, 1914.....	9.90	70.6	6.70	71.3	8.17	74.5	9.08	73.2
August, 1914....	10.76	70.3	8.38	70.4	9.10	73.3	8.26	73.5
September, 1914.	7.53	64.0	5.86	65.6	7.75	68.5	6.54	72.7
October, 1914....	6.08	54.0	3.46	57.7	5.04	61.7	4.47	71.5
November, 1914..	3.28	48.9	2.95	53.3	3.62	57.3	4.34	68.7
December, 1914..	3.94	35.2	1.38	41.2	1.65	41.6	2.57	55.6
	79.55	53.5	54.22	56.6	61.94	58.8	65.87	66.6
January, 1915....	3.47	38.5	1.75	42.4	1.96	43.8	1.73	57.0
February, 1915..	3.97	39.6	(3.00)	44.6	2.51	45.1	2.40	57.7
March, 1915.....	4.56	46.3	4.62	49.2	3.16	50.6	4.99	63.7
April, 1915.....	3.75	47.8	4.85	53.3	4.44	52.9	5.15	63.5
May, 1915.....	4.52	50.0	4.45	55.4	6.17	55.7	6.45	65.5
June, 1915.....	9.66	65.0	9.21	63.5	9.68	66.6	8.01	71.6
July, 1915.....	8.74	68.5	10.45	68.0	10.46	68.4	9.34	74.8
August, 1915.....	10.30	69.8	8.70	71.5	8.28	75.2	7.89	77.5
September, 1915.	6.88	61.7	7.11	63.0	8.90	68.0	6.02	72.7

NOTE.—Figures in parentheses are interpolated from other pans.

* Mean temperature from Warners Spring, 4 miles east.

Mr. Post.

TABLE 52.—GROSS EVAPORATION RECORDS. 3-FT. LAND-PANS, EXCEPT AS NOTED. VOLCAN LAND AND WATER COMPANY.

ELEVATION.	4 640 FT.		2 800 FT.		2 800 FT.		900 FT.	2 100 FT.	1 400 FT.	230 FT.	
	Pan No. 2.		Pan No. 6‖ (*E*).		Pan No. 6-*A*.		Pan No. 8.	Pan No. 9.	Pan No. 10.	Pan No. 11.	
Month.	Evaporation, in inches.	Mean temperature.*	Evaporation, in inches.	Mean temperature.†	Evaporation, in inches.	Mean temperature.†	Evaporation, in inches.	Evaporation, in inches.§	Evaporation, in inches.	Evaporation, in inches.	Mean temperature.‡
1913											
January...	.....	34.8	(2.00)	42.9	.....	.....	.....	.....	.....	.....	.....
February..	.....	37.5	(3.00)	44.4	.....	.....	.....	.....	.....	.....	.....
March.....	.....	39.2	3.58	47.5	.....	.....	.....	.....	.....	.....	.....
April	4.87	48.6	4.97	52.6	.....	.....	.....	.....	.....	.....	.....
May.....	8.38	56.1	4.96	59.3	.....	.....	.....	.....	.....	.....	.....
June.....	8.90	59.0	4.66	63.9	.....	.....	.....	.....	.....	.....	.....
July.....	8.99	66.6	6.67	71.6	.....	.....	.....	.....	.....	.....	.....
August....	8.96	68.2	5.85	74.2	.....	.....	.....	.....	.....	.....	.....
September	9.39	65.4	6.73	71.3	.....	.....	.....	.....	.....	.....	.....
October...	7.49	54.8	4.90	61.7	.....	.....	.....	.....	.....	.....	.....
November.	4.40	45.1	2.82	51.7	.....	.....	.....	.....	.....	.....	.....
December.	2.30	38.5	1.93	45.0	.....	.....	.....	.....	.....	.....	.....
	63.68	51.2	52.07	57.2	.....	.....	.....	.....	.....	.....	.....
1914											
January...	2.30	42.4	1.70	47.7	.....	.....	2.79	1.58	3.03	.....	.....
February..	3.32	42.0	2.67	49.0	.....	.....	2.56	2.43	2.63	.....	.....
March.....	5.26	47.4	3.49	54.2	.....	.....	3.51	3.33	4.03	.....	.....
April.....	5.50	49.4	3.07	55.2	.....	.....	3.56	3.58	4.88	.....	.....
May.....	6.19	53.9	3.30	58.9	.....	.....	4.72	3.73	4.58	.....	.....
June.....	10.04	63.4	4.34	63.7	.....	.....	8.23	6.06	8.83	.....	.....
July.....	12.23	70.6	5.05	74.5	.....	.....	10.26	7.65	8.77	.....	.....
August....	12.42	70.3	4.97	73.3	.....	.....	9.92	7.83	8.23	.....	.....
September.	8.93	64.0	4.53	68.5	.....	.....	6.68	5.78	6.77	.....	.....
October....	6.76	54.0	4.36	61.7	.....	.....	4.51	6.52	5.49	.....	.....
November.	4.20	48.9	3.42	57.3	.....	.....	3.16	4.42	4.69	.....	.....
December..	1.72	35.2	1.87	41.6	.....	.....	1.40	3.07	1.85	.....	.....
	78.87	53.5	42.77	58.8	.....	.....	61.30	55.98	63.78	.....	.....
1915											
January...	2.93	38.5	1.63	43.8	.....	.....	1.44	1.44	1.61	1.07	48.9
February..	3.12	39.6	2.70	45.1	.....	.....	2.00	.....	2.42	1.15	51.2
March.....	4.77	46.3	3.02	50.6	.....	.....	3.07	3.01	3.33	3.06	57.5
April.....	4.29	47.8	4.25	52.9	.....	.....	5.29	3.09	3.00	2.45	59.8
May.....	4.92	50.0	.....	.....	4.73	55.7	4.00	3.65	3.75	3.92	62.1
June.....	7.94	65.0	.....	.....	7.86	66.6	6.18	5.36	5.83	5.89	69.1
July.....	10.22	68.5	.....	.....	12.01	68.4	6.52	6.02	6.62	6.83	72.8
August....	11.14	69.8	.....	.....	9.89	75.2	6.62	6.08	6.93	6.77	73.6
September.	7.80	61.7	.....	.....	9.41	68.0	5.01	4.50	5.00	4.90	.....

* Mean temperature at Pan No. 1.

† Mean temperature from Warners Springs, 4 miles east.

‡ Mean temperature from Escondido, 5 miles northeast.

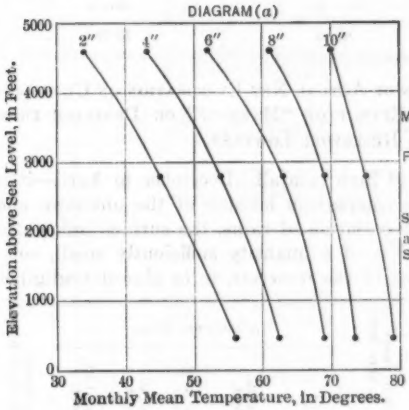
§ 9-in. square pan, January to November, inclusive, 1914. 3-ft. square, standard pan, December, 1914 and 1915.

‖ Pan No. 6 in moist spring-fed ground discontinued.

The annual totals are incorporated in the summary, Table 53, under the captions "Sweetwater Reservoir" and "Independence". Mr. Post.

"FULL-SCALE" DATA OF ACTUAL GROSS RESERVOIR EVAPORATION.

In the application of such data to hydrographic studies, it is clear that the ratio which a pan bears to a "full-scale" reservoir should be determined. This discussion may be considered as a progress report on this ratio.



MONTHLY DEPTHS OF EVAPORATION FOR SOUTHERN CALIFORNIA FROM 3-FT. SQUARE FLOATING PANS, MEAN TEMPERATURES AND ELEVATIONS.

Showing the results of the Volcan Land and Water Company's Observations in San Diego County, California.

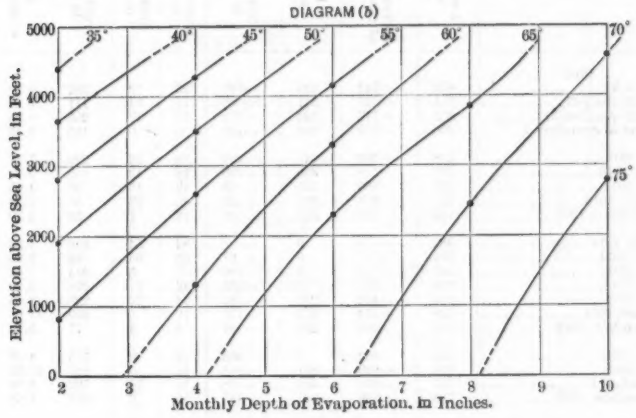


FIG. 18.

Mr. Post.

TABLE 53.

Year.	Independence. Altitude = 3 760 ft.	Sweetwater. Altitude = 200 ft.
1889	.....	57.35
1890	.....	59.85
1891	.....	58.08
1892	.....	59.55
1909	69.05	.....
1910	65.44	.....
1912	.....	.....
1913	.....	.....
1913	.....	.....
1914	.....	.....
Averages	67.25	58.70

TABLE 54.—DETERMINATION OF ACTUAL NET EVAPORATION ON CUYAMACA LAKE (INCLUDING THE EFFECT OF "MAKE-UP" OR DRAINAGE FROM MOIST BORDER LAND AS RESERVOIR LOWERS).

NOTE.—During the months of high rainfall—December to April—it is impossible to make this comparison because of the unknown and large run-off. With the cessation of rains, the surface inflow can be closely observed, and is of a quantity sufficiently small, compared with the large area of the reservoir, to be almost negligible.

Period.	Average number of acres exposed.	Increase + or decrease − of reservoir, in acre-feet.	ACCOUNTED FOR: Draft, in acre-feet.	Net depletion, in acre-feet.	Run-off into lake, in acre-feet.	Direct rainfall, in acre-feet.	Gross evaporation, in acre-feet.	Monthly gross evaporation, in inches.
1912.								
June 1–30	472	−542	195	−347	23	12	382	9.68
June 30–July 27	414	−627	541	− 86	11	14	111	3.57
July 27–August 31	332	−747	523	−224	14	17	255	8.39
August 31–October 1	250	−419	290	−129	6	0	135	6.48
May, 1913	418	−263	47	−216	24	12	242	7.27
June, 1913	361	−518	385	−133	18	12	163	5.24
July, 1913	283	−600	531	+ 69	19	15	98	4.05
August, 1913	220	−353	381	+ 28	45	33	50	2.78
September, 1913	145	−369	329	− 40	12	1	53	4.39
March, 1914	380	− 8	0	− 8	84	54	146	4.63
April, 1914	384	+ 67	0	+ 67	68	63	64	2.00
May, 1914	380	−199	0	−199	20	5	224	7.08
June, 1914	356	−255	0	−255	12	11	278	9.30
July, 1914	285	−452	280	−187	14	22	223	8.85
August, 1914	228	−472	275	−197	12	0	209	11.00
September, 1914	155	−282	215	− 67	6	4	77	5.98
June, 1915	844	−563	0	−563	12	0	575	8.16
July, 1915	816	−829	131	−698	12	0	816	10.30
August, 1915	777	−860	422	−438	19	75	532	8.28
September, 1915	738	−836	391	−445	12	0	457	7.48

Mr. Post.

TABLE 55.—CORRECTION OF SWEETWATER EVAPORATION DATA.

TO REMOVE 5% SYSTEM LOSS INCLUDED IN EVAPORATION AS GIVEN BY JOHN F. COVERT, ASSOC. M. AM. SOC. C. E.

Quantities, in feet.

Month.	1910.			1911.			1912.			1913.			1914.		
	As given by Covert.	Losses 5%.	True evaporation.	As given by Covert.	Losses 5%.	True evaporation.	As given by Covert.	Losses 5%.	True evaporation.	As given by Covert.	Losses 5%.	True evaporation.	As given by Covert.	Losses 5%.	True evaporation.
January	0.200	0.001	0.199	0.260	0.006	0.257	0.130	0.002	0.128	0.356	0.015	0.341	0.148	0.008	0.140
February	0.200	0.003	0.197	0.200	0.001	0.199	0.330	0.021	0.309	0.195	0.006	0.189	0.204*	.....	0.174
March	0.270	0.009	0.261	0.270	0.002	0.268	0.170	0.016	0.154	0.266*	.....	0.226	0.266*	.....	0.227
April	0.320	0.009	0.311	0.350	0.032	0.318	0.480	0.004	0.476	0.347*	.....	0.295	0.347*	.....	0.295
May	0.650	0.111	0.539	0.610	0.047	0.563	0.410	0.015	0.395	0.651	0.076	0.578	0.531	0.063	0.468
June	0.590	0.036	0.554	0.630	0.046	0.574	0.740	0.059	0.681	0.344	0.065	0.279	0.811	0.093	0.718
July	0.720	0.038	0.682	0.670	0.049	0.621	0.680	0.056	0.624	1.053	0 076	0.977	0.785	0.111	0.674
August	0.610	0.027	0.583	0.660	0.055	0.605	0.760	0.068	0.692	0.718	0.092	0.626	0.734	0.134	0.600
September	0.440	0.028	0.412	0.690	0.045	0.645	0.600	0.060	0.540	0.674	0.096	0.578	0.328	0.158	0.170
October	0.510	0.017	0.493	0.500	0.044	0.456	0.460	0.049	0.411	0.459	0.078	0.381	0.368	0.100	0.268
November	0.360	0.013	0.347	0.380	0.032	0.348	0.370	0.046	0.324	0.310	0.042	0.268	0.084	0.072	0.012
December	0.380	0.000	0.380	0.340	0.018	0.322	0.080	0.031	0.049	0.106	0.018	0.088	0.089	0.021	0.068
Totals	5.25	0.292	4.958	5.55	.....	5.176	5.21	.....	4.783	5.482	.....	4.826	.....	4.695	3.814

* Pan records used: revised by taking 85% of pan records.

Mr. Post.

Cuyamaca Reservoir.—It was hoped that a comparison of the reservoir stages at Cuyamaca Lake, accounting for inflow and draft, would establish this ratio. A decided practical difficulty has been met in considering the "make-up" of the reservoir, which is very flat, and a slight change in level undoubtedly drains a large area of moist lands. However, it serves as an example of the actual net evaporation, including "make-up". This is somewhat compensated by seepage from the dam.

In so far as rising or stationary conditions are incorporated in the calculation, "make-up" is neutralized. It is neglected as a factor when averaging the 13 months of observations.

Sweetwater Reservoir.—John F. Covert, Assoc. M. Am. Soc. C. E., Engineer of the San Diego Land and Town Company, has computed the gross evaporation of Sweetwater Reservoir, making all allowances except for losses in the distribution system, as he was obliged to use the summation of the meters of the distribution system to obtain the draft.

This distribution loss is assumed by the writer to be 5%, and Mr. Covert's data have been revised to this extent.

TABLE 56.—SWEETWATER GROSS EVAPORATION, IN INCHES (FINALLY CORRECTED).

Month.	1910.	1911.	1912.	1913.	1914.	Mean.	Percentage.
January	2.39	3.09	1.54	4.10	1.68	2.56	4
February	2.36	2.39	3.70	2.26	2.09	2.56	5
March	3.15	3.21	1.85	2.71	2.73	2.73	5
April	3.74	3.89	5.71	3.54	3.54	4.08	7
May	6.43	6.78	4.75	6.91	5.60	6.09	11
June	6.68	6.90	8.19	3.35	8.60	6.74	12
July	8.20	7.48	7.50	11.70	8.10	8.80	15
August	7.00	7.26	8.31	7.50	7.20	7.45	13
September	4.94	7.73	6.49	6.91	2.04	5.62	10
October	5.91	6.65	4.94	4.58	3.21	5.06	9
November	4.16	4.17	3.89	3.21	0.14	3.11	5
December	4.56	3.86	0.59	1.06	0.83	2.18	4
Totals	59.52	63.32	57.46	57.83	45.76	56.98	100

Other "Actual" Comparisons.—On Murray Hill Reservoir, near Grossmont, valves were closed, and there was no inflow during the period from April 26th 3.30 P. M. to 7.30 A. M. May 2d, 1913.

Drop in lake.....1.60 in.
The rate is.....1.60 = 0.282 in. per day.

5.68

Mr. Post.

At La Mesa, floating-pan No. 7, 5 miles distant, during the same period, the decrease of water in the pan from April 25th (noon) to May 2d (noon).....1.76 in.

Subtract 1⅓ days excess of period at rate of 0.200 per day, as per preceding week at La Mesa.....0.27 "

1.49 in.

$$\frac{\text{Actual}}{\text{Pan}} = \frac{1.60}{1.49} = 107 \text{ per cent.}$$

In this instance the lake shows a greater evaporation than the pan.

Upper Otay Reservoir.—This is a small water-shed with practically no run-off during the months given, and no draft, except some domestic use on the Babcock place, which is not known, but probably not great.

TABLE 57.—OBSERVATIONS BY CITY OF SAN DIEGO CARETAKERS.
(Probably rather roughly to the nearest inch.)

Month.	Fall in reservoir, in inches.	Rainfall,† in inches.	Evaporation, in inches.
June, 1914.....	6.64*	0	6.64
July, "	8.75	0	8.75
Aug., "	12.00	0.06†	12.06
June, 1915.....	6.00	0	6.00
July, "	8.00	0	8.00
Aug., "	10.00	0	10.00
Sept., "	5.00	0	5.00

* Interpolated 1 day.

† At Sweetwater Reservoir, 5 miles distant.

TABLE 58.—DETERMINATION OF THE RATIO BETWEEN ACTUAL EVAPORATION AND FLOATING-PAN.

Month.	Cuyamaca Reservoir. Actual as computed from Table 54.	Floating-pan No. 1.	Ratio.
June, 1913.....	5.24	9.25	57%
July, 1913.....	4.05	8.18	49%
August, 1913.....	2.78*	7.94*	35%
September, 1913.	4.39	9.68	45%
March, 1914.....	4.63	7.20	64%
April, 1914.....	2.00*	5.60*	36%
May, 1914.....	7.08	7.53	94%
June, 1914.....	9.30	8.99	103%
July, 1914.....	8.85	9.90	89%
August, 1914.....	11.00	10.76	102%
September, 1914.	5.98	7.53	79%
June, 1915.....	8.16	9.66	84%
July, 1915.....	10.30	8.74	118%
August, 1915.....	8.28	10.30	80%
September, 1915.	7.48	6.88	108%
	91.74*	114.20*	83%

* August, 1913, and April, 1914, excluded from totals.

Mr. Post.

TABLE 59.—DETERMINATION OF THE RATIO BETWEEN ACTUAL EVAPORATION AND FLOATING-PAN.

Sweetwater Reservoir (Table 56) and Upper Otay Reservoir (Table 57), compared with Pan No. 7 floating on La Mesa Reservoir. These three localities are roughly the same distance back from the Pacific Coast line, respectively, 12, 9, and 12 miles; the altitudes are 200, 500, and 480 ft. Pan No. 7 is the most northerly, Sweetwater Reservoir is distant 7 miles, and Upper Otay Reservoir 5 miles farther.

Month.	Actual.			Pan No. 7.
	Sweetwater.	Upper Otay.	Mean.	
January, 1913	4.10	.....	4.10	1.59
February, 1913	2.26	.....	2.26	4.23
March, 1913	2.71	.....	2.71	4.51
April, 1913	3.54	.....	3.54	6.19
May, 1913	6.91	.....	6.91	7.54
June, 1913	3.35	.....	3.35	6.98
July, 1913	11.70	.....	11.70	9.64
August, 1913	7.50	.....	7.50	8.45
September, 1913	6.91	.....	6.91	8.21
October, 1913	4.58	.....	4.58	6.46
November, 1913	3.21	.....	3.21	3.42
December, 1913	1.06	.....	1.06	2.70
January, 1914	1.68	.....	1.68	2.37
February, 1914	2.09	.....	2.09	2.94
March, 1914	2.73	.....	2.73	6.15
April, 1914	3.54	.....	3.54	6.16
May, 1914	5.60	.....	5.60	5.48
June, 1914	8.60	6.64	7.62	7.76
July, 1914	8.10	8.75	8.48	9.03
August, 1914	7.20	12.06	9.63	8.26
September, 1914	2.04	.....	2.04	6.54
October, 1914	3.21	.....	3.21	4.47
November, 1914	0.14	.....	0.14	4.34
December, 1914	0.83	.....	0.83	2.37
June, 1915	.....	6.00	6.00	8.01
July, 1915	.....	8.00	8.00	9.34
August, 1915	.....	10.00	10.00	7.89
September, 1915	.....	5.00	5.00	6.02
	.....	.....	134.42	167.05 Ratio = 80 per cent.

TABLE 60.—DETERMINATION OF THE RATIO BETWEEN FLOATING-PAN AND LAND-PAN. Mr. Post.

Pans were 2 miles apart; elevation = 4 600 ft.

Month.	Pan No. 1. Floating.	Pan No. 2. Land.	Ratio.
June, 1913	9.25	8.90	104%
July, 1913	8.18	8.99	91%
August, 1913	7.94	8.96	89%
September, 1913	9.68	9.39	102%
October, 1913	6.86	7.49	92%
November, 1913	5.63	4.40	128%
December, 1913	2.85	2.30	126%
January, 1914	3.28	2.30	138%
February, 1914	5.46	3.32	146%
March, 1914	7.20	5.26	136%
April, 1914	5.60	5.50	102%
May, 1914	7.53	6.19	122%
June, 1914	8.99	10.04	90%
July, 1914	9.90	12.23	89%
August, 1914	10.76	12.42	87%
September, 1914	7.53	8.93	84%
October, 1914	6.08	6.76	90%
November, 1914	3.28	4.20	78%
December, 1914	3.94	1.72	230%
January, 1915	3.47	2.93	118%
February, 1915	3.97	3.12	127%
March, 1915	4.56	4.77	97%
April, 1915	3.75	4.29	87%
May, 1915	4.52	4.92	92%
June, 1915	9.66	7.94	121%
July, 1915	8.74	10.22	85%
August, 1915	10.30	11.14	91%
September, 1915	6.88	7.80	88%
	185.79	186.43	99%

TABLE 61.—DETERMINATION OF RATIO BETWEEN FLOATING-PAN AND LAND-PANS.

Pans adjacent; elevation, 2 800 ft.

Month.	Pan No. 5.	Pan No. 6-A.	Ratio.
May, 1915	6.17	4.73	130%
June, 1915	9.68	7.86	115%
July, 1915	10.46	12.01	88%
August, 1915	8.28	9.89	83%
September, 1915	8.90	9.41	94%
	43.49	43.90	99%

Mr. Post.

TABLE 62.—DISTRIBUTION OF EVAPORATION IN SOUTHERN CALIFORNIA.

Month.	Sweetwater. (Table 56.)	Sweetwater. (Pans.)	Pan No. 7.	Pan No. 5.	Pan No. 2.
January	4	4	3	4	4
February	5	4	5	5	4
March	5	5	8	7	7
April	7	7	9	8	6
May	11	11	10	10	9
June	12	11	10	10	12
July	15	15	13	12	14
August	13	14	11	12	14
September	10	11	10	12	12
October	9	9	9	10	9
November	5	5	6	6	6
December	4	4	6	4	3
	100%	100%	100%	100%	100%

Mr. Thorpe.

JOHN E. STIRLING THORPE,* ESQ. (by letter).—Soon after Mr. Duryea was retained as Consulting Engineer of the Mexican Northern Power Company, he asked the writer whether any study had been made of the evaporation of Lake Conchos and, if not, it was his wish that one should be commenced immediately. Subsequently, he outlined in detail the sizes, shape, etc., of the pans he wished to have made, and stated where he wanted the land and floating pans placed.

Due to the importance of obtaining accurate measurements, great care was taken in making the evaporation pans, especially the floating pans. The high winds and the wide expanse of the lake caused it to become very rough at times, necessitating a very strong pan. The requisite strength was obtained by making a raft of 6 by 8-in. scantling and placing splash-boards around the sides to prevent, as much as possible, the water of the lake from entering the pan. This type is illustrated by Fig. 19. One side of the splash-boards opened like a sliding door. When the observer wished to take measurements, he opened this door, and was able to run the bow of his boat to within a few inches of the pan, and take his measurements without disturbing its equilibrium. In rough weather, the pan was towed into a near-by cove in order that more accurate measurements could be made. The floating pans were anchored, one in the middle of each of the wide areas of the lake.

The first land pans were placed on a base of 2 by 4-in. scantling, with a strip of heavy lagging nailed around the outside of the pan to prevent the rays of the sun from striking the water. Later, this arrangement was changed by Mr. Duryea, the pans being placed directly on the ground, with earth packed around the sides level with their tops. On account of the large numbers of stray animals in this section of Mexico, special care was taken to prevent them from drink-

* Birmingham, Ala.

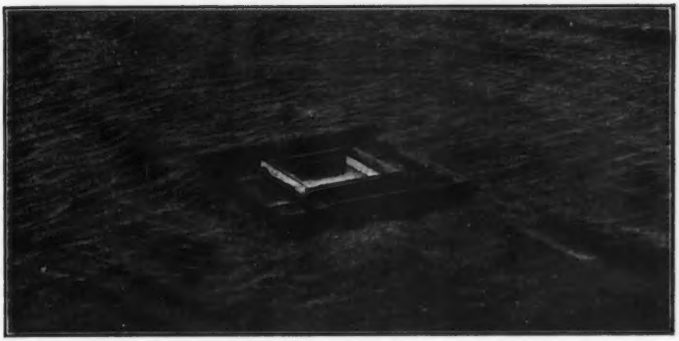


FIG. 19.—TYPE OF FLOATING PAN USED AT LAKE CONCHOS.

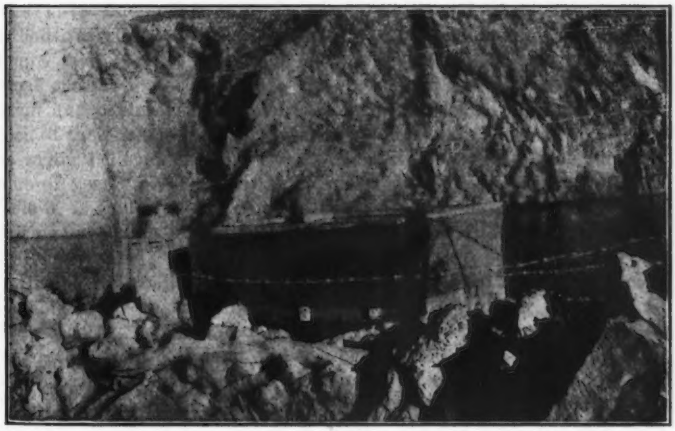
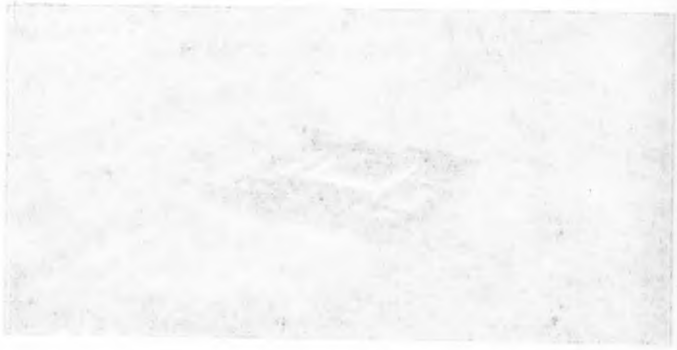


FIG. 20.—TYPE OF LAND PAN USED AT LAKE CONCHOS, AS FIRST PLACED.



ing any of the water in the pans by constructing a strong fence of 6 by 6-in. scantling and barbed wire. Fig. 20 illustrates one of the land pans, as first placed. This pan was set at Elevation 1 296. Another and similar pan was set at Elevation 1 317. Mr. Thorpe.

One of the junior engineers was detailed to take all measurements, keep the pans filled to a depth of approximately 10 in., check up leaks, etc. A watchman also visited the pans several times during the 24 hours to prevent any tampering with them.

A water line was painted on the inside of each corner of the pan, and all four corners were checked up on each measurement of the depth; but the corner marked No. 1 was taken as the master corner, from which all depths were taken.

No trouble was experienced with the land pans, with which great care was taken at all times, and accurate results were obtained. This statement applies also to the floating pans during calm weather; but during rough weather the results from these pans were doubtful and were so treated, careful attention being paid to weather conditions. Four engineers, including the writer, took independent observations each day, and the majority opinion of these observations was briefly recorded each day with the temperature observations taken with maximum and minimum thermometers. At each land pan there was a rain gauge, and this was observed jointly with the pan.

CHARLES H. LEE,* ASSOC. M. AM. SOC. C. E. (by letter).—The authors have attempted, through assumptions and highly theoretical methods, carried out in great detail, to estimate annual evaporation from a lake surface more than 300 miles from the nearest point of actual observation. They have also attempted an exhaustive investigation and comprehensive analyses of the problem. They have based much of their framework, however, on a fundamental assumption which the writer believes to be in error. As a result thereof, the computed value of yearly gross evaporation from Lake Conchos, as set forth in Table 4, is far from the truth. The paper will be discussed by considering first each of the subsidiary conclusions and finally the principal conclusion. Mr. Lee.

During the past 10 years, the writer has had occasion to give considerable study to the subject of evaporation. During the 3 years, 1908 to 1911, he carried on extensive quantitative observations of evaporation from both land and soil pans in Owens Valley, California, and subsequently has made a computation of annual evaporation from Owens Lake, based on detailed observations of inflow, lake level, and lake area, covering a period of 8 years. He has kept in touch with the experiments conducted for the United States Weather Bureau at Reno and Salton Sea, by Professor Frank H. Bigelow, and has also

* Los Angeles, Cal.

Mr. Lee. made a study of the various available records in Western States, both published and unpublished.

The subsidiary conclusions will be taken up in order as follows.

(*a*)—*Relative Evaporation Depths from Pans of Different Sizes.*—The writer wishes to point out, in connection with Professor Bigelow's formula,

$$E_0 = C_2 \times \frac{e_s}{e_a} \times \frac{d\,e}{d\,S} \times (1 + 0.070\,w),$$

that it represents the evaporation depths from isolated areas of exposed water surface distantly removed from similar areas. It not only represents evaporation depth from pans, but also from open tanks, pools, ponds, lakes, or seas. The coefficient, C_2, is a variable, depending on the size of the evaporating surface. Small pans, representing one extreme, give large values of C_2. Larger pans give rapidly decreasing values of C_2. Pools and ponds would give less rapidly decreasing values, and large lakes give practically the smallest value, or approximately 0.024. In brief, the equation does not give evaporation depths from pans, only, as one might infer from the authors' statements and from Fig. 5, but from the surfaces of isolated bodies of water of various sizes.

Carrying the idea a little further, the writer would emphasize the fact that the equation does not apply to a pan floating on a lake surface, except as it applies to that surface as a whole. Water isolated in a pan floating on a lake surface is losing by evaporation under the same vapor blanket conditions as the surrounding water surface outside the pan. There may be a difference of temperature between the water inside and outside of such a pan, because of the absorption by the metal of heat from the sun's rays. The writer has observed such temperature differences to be very small, however, and Professor Bigelow states that there is "a small correction due to this difference of temperature".*

The coefficient, C_2, depends on the ease with which saturated vapor from a water surface escapes into the atmosphere. In the case of a small pan not within the vapor blanket of a lake, nothing opposes the dissipation of the vapor in four horizontal directions as well as vertically. In the case of a pan floating on a lake, the lateral expansion of the vapor from the pan is opposed in all directions by the vapor arising from the surrounding water, and the only direction of free dissipation is vertically. A lake, considered as a whole, has not as great an opportunity for lateral dissipation of its vapor as a pan, for the reason that vapor blankets do not increase in depth in proportion to the area of the water surface. For this reason, the value of C_2 decreases with the increase in size of the area of water surface.

* "A Manual for Observers in Climatology and Evaporation", by Frank H. Bigelow, U. S. D. A., Weather Bureau No. 409, p. 36.

Mr. Lee.

In passing, the writer draws attention to the fact that Professor Bigelow's coefficients are not for square pans, as the authors state, but for circular pans.*

(*b*)—*Relative Evaporation Depths from Land Pans and from Near-by Floating Pans.*—The writer's experience generally confirms the conclusion of the authors as set forth in Table 8. Thus, at the soil pan near Independence, the evaporation, from April, 1910, to March, 1911, was 85.06 in., as compared with 63.74 in. from a similar pan floating on the surface of Owens River not more than 2 miles away. The depth of evaporation from the floating pan was 75% of that from the soil pan. This relation is, to a certain extent, a special case of the preceding conclusion. The floating pan has practically the same evaporation rate as the large water surface, and the land pan has a greater rate, due partly to its being very much smaller than the lake, and partly to the higher day temperature of the water. The ratio, therefore, is not a fixed one, but varies with the size of the lake, the size of the land pan, and the difference in the water temperatures in the land pan and floating pan.

(*c*)—*Relative Evaporation Depth from Large Reservoirs, as Compared with That from 3-Ft. Square Pans Floating Thereon.*—The authors state (on page 1855) that:

"* * * it has been recognized more or less generally that the evaporation losses from large reservoirs * * * are materially less than the evaporation depths measured in pans, even when the pans are floating on the reservoir."

The writer does not agree with this statement, except in the case of land pans, as previously explained. In none of the quotations from Professor Bigelow is the statement made or inferred that the evaporation from floating pans differs materially from that of the water surface on which they are floating. The assumption by the authors (on page 1857) that the values of C_2, as given by Professor Bigelow, are for floating pans, is not substantiated in any of the latter's writings. None of the pans used at Salton Sea was floating, and it was only there that Professor Bigelow had the opportunity to develop experimentally values for C_2, as published in 1910. The computation by the authors merely means that the evaporation from a large water surface is 61.7% of that from a 3-ft. circular (not square) pan outside of the vapor blanket.

On the same page the authors make the assumption that "the evaporation depths from pans 2 ft. above the surface of the [Salton] sea were the same as if the pans had been floating in the sea instead". The reverse of this statement occurs in numerous places in Professor

* "Provisional Statement Regarding the Total Amount of Evaporation by Months at 23 Stations in the United States, 1909–10," by Professor F. H. Bigelow, U. S. D. A., Weather Bureau (Abstract of Data No. 4).

Mr. Lee. Bigelow's writings. For example, in the second quotation on page 1856 he states:

"If 70 ins. is admitted as the amount evaporated from Salton Sea, there remains 38 ins. as the difference between the [evaporation from the] water in the sea and that in the lower swinging pan [2 ft. above the surface of the sea]."

Again, he says, commenting on the work at Reno*:

"On the other hand there is a decided drop in the value of C_d, from 0.070 to 0.056, in passing from pan 3 to pan 1, that is to the surface of the water in the reservoir. It follows that the evaporation at the surface of the water body is $\frac{0.055}{0.070} = 0.79$ of that at a few feet above the surface. This same difference appears on the two-pan stands at Indio and Mecca, when the upper pan is evaporating 1.27 times faster than the one at the surface, after having applied the other functions of the formula."

The authors' unsupported assumption, therefore, is not in agreement with the exhaustive observations of Professor Bigelow, and hence the result of their computation near the bottom of page 1857 is seriously open to question.

The computations made by the authors on pages 1748-1749, based on data of evaporation from Salton Sea as determined by F. T. Robson, Assoc. M. Am. Soc. C. E., and C. E. Grunsky, M. Am. Soc. C. E., from lake level and inflow, involve the assumption, already shown to be incorrect, that the relation of evaporation from floating pans of various sizes differs from that from the surface of the body of water in which they float, and vary as do land pans of similar size. The result of the computation, therefore, is without meaning.

The writer believes that this discussion shows the final conclusions of the authors, on pages 1860 and 1862, to be fallacious, namely, that, in general, for large reservoirs, or at Lake Conchos, the yearly evaporation depth is 62% of the evaporation depth from a 3-ft. square pan floating thereon. On the other hand, he maintains that, with possibly a slight error, due to difference in temperature of the water, these evaporation depths are essentially the same.

(*d*)—*Relations Between Evaporation Depth and Mean Temperature.*—The authors' analysis of this subject is very interesting and suggestive, and the results are of value within the general region to which they can be applied and when evaporation records are not available. The writer places little confidence in the Piche evaporimeter records used by the authors. He has had occasion, in several instances, to check them up with subsequent floating-pan records, and has found the results widely divergent. The authors have used Piche

* *Monthly Weather Review*, Annual Summary, 1908, pp. 437–445.

evaporimeter calculations at eleven of the seventeen points in New Mexico and Texas for which evaporation records are given in Table 17. Mr. Lee.

(e)—Relations Between Evaporation Depth and Elevation Above Sea.—The writer would make much the same comment on this section of the paper as he has on the preceding one. As of interest in connection with the subject, attention is called to the diagram, Fig. 9,* prepared by himself from observations by the Office of Experiment Stations, U. S. Department of Agriculture, showing the combined influences of altitude and temperature on evaporation on the east slope of Mt. Whitney, Inyo County, California.

The authors' principal conclusion will now be considered.

Method A.—The writer, as he has already explained at length, does not agree with the authors in the application of the correction factor of 62% to floating-pan evaporation depths, and therefore does not concur in their result of 52.5 in. as the annual evaporation at Lake Conchos. He does not mean to infer by this that he believes 84.74 in. represents the actual value. He does believe, however, that for all practical engineering purposes, floating-pan observations give the true depth of evaporation from a large body of water.

Methods B_a and B_b.—Both these methods involve the application of the factor, 62%, and, to that extent at least, the writer believes the results are in error.

Method C.—This method is the only one used by the authors which does not involve the correction factor, 62%, for floating pans. The results apparently agree closely with those obtained by the other methods. The writer does not place sufficient confidence in this method, however, to regard it as a check on the others. In the first place, the proportion of the annual evaporation depth assumed to occur during the 4 months, October to January, and the 7 months, October to April (Table 46), is based entirely on the application of temperature-evaporation studies in New Mexico and Texas to temperature conditions at Lake Conchos. Almost two-thirds of the New Mexico and Texas evaporation records, as previously shown, are Piche evaporimeter computations. Furthermore, the months under consideration are those of smallest evaporation loss, and a change of a few per cent. would make a great difference in the computed annual evaporation. For example, if the 4 months, October to January, were 17% instead of 21.7%, the result would be 71 in., instead of 55.4 in. At Independence, Cal., at Elevation 3 800, 500 ft. lower than Lake Conchos, the measured evaporation from a floating pan during these months is 14% of the annual.† Finally, a reservoir or lake, particularly one as recently flooded as Lake Conchos, does not show as great regularity of monthly

* *Transactions*, Am. Soc. C. E., Vol. LXXVIII, p. 186.

† *Water Supply Paper No. 294*, p. 118.

Mr. Lee.

variation in evaporation computed from inflow, outflow, and lake-level data, as pan observations. The writer has at hand accurate inflow and lake-level measurements at Owens Lake, California, for 8 years, and although the computed annual depths of evaporation agree closely, the monthly variation is very erratic, and agrees only in a very general way with a floating-pan record. Therefore, he does not believe that sufficient weight can be placed on Method *C* to regard it as supporting the other methods.

In concluding this discussion, the writer draws attention to what the authors state is the most important feature of their paper (page 1831), namely,

"* * * it is estimated by a combination of the results of Methods *B* and *C* that at Lake Conchos the evaporation depth from the lake surface certainly is less than 67.5% (and in all probability as little as 62%) of that from a pan 3 ft. square floating thereon. In 1910 Professor Frank H. Bigelow, of the United States Weather Bureau (in connection with his Salton Sea evaporation experiments), deduced a value of 62% for this coefficient; and, so far as known to the writers, the foregoing value at Lake Conchos is the only check of Professor Bigelow's value which ever has been made."

The writer maintains, first, that the Lake Conchos observations did not prove, conclusively or even tentatively, that the evaporation from a large water surface differs appreciably from a properly installed pan floating thereon; second, that Professor Bigelow's work at Salton Sea did not prove, or even attempt to prove, that such a difference existed; and third, that, in these respects, the authors, throughout their paper, have labored under a misconception.

As positive proof of the writer's contention, he will state that the annual evaporation from a pan floating on the surface of Owens River, near Independence, Cal., was observed by him to be 67.2 in. averaged for two years;* that the annual evaporation from Owens Lake, which has no outlet, is 20 miles south of Independence, and has practically the same climatic conditions, is 61.0 in., as computed from accurate records of inflow and lake level for 8 years, together with a detailed area curve of the lake basin; and that exhaustive experiments conducted with fresh water and Owens Lake brine, the latter having an average specific gravity of 1.11, during the 8-year period, show that the evaporation rate from Owens Lake water at that specific gravity is 10% less than from fresh water. Hence, the evaporation from the floating-pan checks with that from Owens Lake to less than 1 per cent.

Mr. Follansbee.

ROBERT FOLLANSBEE,† ASSOC. M. AM. SOC. C. E. (by letter).—Records at Santa Fé, N. Mex., from a 3-ft. floating-pan, show that the Piche method of determining evaporation gives results too great. The Piche

* *Water Supply Paper No. 294*, p. 118.

† Denver, Colo.

Mr. Follansbee.

records at Santa Fé give 79.8 in., and actual measurements from April, 1913, to September, 1914, show a mean of 59.2 in. On the other hand, the lines of equal evaporation on Fig. 3 show approximately 50 in. for Southwestern Oklahoma, though the floating-pan records, from February, 1913, to October, 1915, show a mean of 61 in., approximately.

On page 1841 the authors state that the "adopted evaporation depth evidently is a gross depth; and, for use in the water-supply investigation, requires to be reduced by the average yearly depth of rainfall on the lake surface." If the discharge is measured at the point of inflow, this statement is correct, but if it is measured at or below the point of outflow, the rainfall should not be deducted, as it is already taken into account by the increased flow which it causes at the measuring point. Also, in water-supply investigations involving storage in a natural lake, evaporation does not enter into the calculation at all, if the discharge is measured at the outlet, unless the storage contemplated will increase appreciably the water surface of the lake, and then the evaporation should be applied only to the increased area, as that from the normal or natural water surface has already taken place, and the measured outflow shows the run-off less such evaporation.

The authors' conclusion that the evaporation from a reservoir surface is somewhat less than 67.7% of the results obtained by a standard 3-ft. pan floating on the water, is of extreme importance in studies of storage, especially in the western sections of the United States. In the extensive studies of evaporation carried on by the Weather Bureau at Salton Sea, the coefficient, 62%, was obtained by comparison with records from a pan placed on a tower 2 ft. above the water surface. As these studies showed that the rate of evaporation depended on the distance the pan was placed above the water surface, and as Professor Bigelow, in his discussion of evaporation,* states that "evaporation is very sensitive to the atmospheric conditions within an inch or two of the surface of the water and to the temperature of the water," the writer, for one, was not convinced that the coefficient, 62%, would apply to records from floating-pans. For that reason, the conclusions reached by the authors are especially valuable. By comparison with the Salton Sea experiments, they seem to indicate that the rate of evaporation is nearly constant within a vertical distance of at least 2 ft. from the water surface itself, instead of being confined to a distance of a few inches, as was indicated by Professor Bigelow in his discussion quoted previously.

A contributing factor to the greater evaporation in floating-pans is the wetted strip around the inside of the pan, just above the normal

* "A Manual for Observers in Climatology and Evaporation."

Mr. Follansbee.

water surface, caused by the wave action within the pan itself. This frequent wetting of the sides of the pan exposes an additional wetted area on a warm iron surface, and hence causes greater evaporation.

The writer was especially interested in the simple relations between evaporations and temperature for the records in the southern part of the Rio Grande drainage. To determine whether a similar relation exists for the upper part of the Rio Grande drainage at an elevation of 7 200 ft., the records near Santa Fé were investigated, and were compiled by the U. S. Geological Survey.* The equipment at the station consisted of a standard pan, 3 ft. square and 18 in. deep, which was floated on a small reservoir 1 mile west of Santa Fé (elevation 6 900 ft.) until May 1st, 1914, when the station was moved to the reservoir of the Santa Fé Water and Light Company, 2 miles east of Santa Fé (elevation 7 200 ft.). The record is given in Table 63.

TABLE 63.—TOTAL EVAPORATION NEAR SANTA FÉ, N. MEX.

Month.	1913.		1914.	
	Evaporation, in inches.	Temperature* of air.	Evaporation, in inches.	Temperature* of air.
Jan.	....	....	1.96	33.5
Feb.	....	....	2.20	32.6
Mar.	....	....	3.85	39.4
Apr.	....	....	5.66	48.0
May.	8.36	58.6	5.95	56.2
June.	6.65	62.5	10.19	67.0
July.	7.23	69.6	6.24	67.0
Aug.	6.37	68.6	5.97	66.8
Sept.	5.54	57.2	6.28	63.0
Oct.	5.15	47.8	....	....
Nov.	3.29	42.4	....	....
Dec.	2.66	26.4	....	....

*Taken from Weather Bureau records at Santa Fé.

The results were first plotted by using water temperatures, but, as the relation between evaporation and temperature is chiefly of value when used in determining evaporation when air temperatures only are available, the results were replotted, using the air temperatures recorded at Santa Fé. This latter plotting, Fig. 21, shows that the points do not lie on or near a single straight line, as would be the case if the simple relation between monthly evaporation and temperature obtained. The points divide into two more or less distinct groups, one defining perfectly a line indicating evaporation nearly 30% greater than that shown by most of the remaining points. It is worth noting that the points defining the line of higher evaporation represent spring months chiefly, when it would be expected that the evaporation for a given air temperature would be less on account of the lower temperature of the water. From these records, it is evident that other

* *Water Supply Papers Nos. 358* and *388*.

factors than temperature—such as wind velocity and relative humidity —influence evaporation to such an extent in the upper part of the Rio Grande drainage that it is impossible to determine evaporation with any degree of accuracy from temperature records alone. Mr. Follansbee.

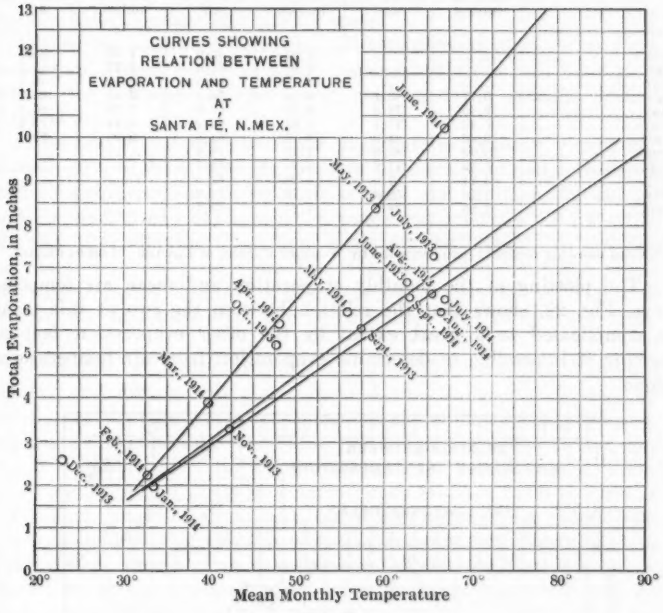


FIG. 21.

An investigation of evaporation records in the southwestern part of Oklahoma was made in order to determine whether a simple relation obtained between temperature and evaporation in that section of the country. These records, which are compiled by the U. S. Geological Survey, are from a standard pan, 3 ft. square and 18 in. deep, floating on the surface of Lake Lawtonka, 14 miles northwest of Lawton, at an elevation of about 1 200 ft.

Table 64 shows the records.*

It may be stated that, owing to the wave action on Lake Lawtonka, it has been found necessary to shorten the upright needle by which the evaporation is measured, so that the water surface in the pan is about 7 in. below the top.

* Published chiefly in *Water Supply Papers Nos. 357* and *387*.

Mr. Follansbee

TABLE 64.—TOTAL EVAPORATION NEAR LAWTON, OKLA.

Month.	1913.		1914.		1915.	
	Evaporation, in inches.	Temperature* of air.	Evaporation, in inches.	Temperature* of air.	Evaporation. in inches.	Temperature* of air.
Jan.	.....	.....	1.94	45.6	1.39	38.0
Feb.	.....	.....	1.88	37.8	1.77	45.2
Mar.	.....	.....	4.13	49.5	2.71	40.5
Apr.	5.74	63.1	4.24	56.8	3.15	62.0
May	6.45	72.2	4.31	65.6	7.56	66.3
June	6.43	77.1	7.84	80.2	9.44	75.2
July	9.43	84.6	10.79	84.3	9.90	79.0
Aug.	10.92	85.8	7.56	78.8	.....	.....
Sept.	6.73	71.2	6.96	74.7	.....	.....
Oct.	4.24	57.0	4.28	62.0	.....	.....
Nov.	1.24	52.0	2.72	54.2	.....	.....
Dec.	1.25	38.2	1.21	33.1	.....	.....

* Mean monthly temperature recorded by Weather Bureau at Lawton, 14 miles distant.

The plotting of the monthly evaporation and mean air temperature, Fig. 22, shows that the relation between the two at this point is complicated to a great extent by the other evaporation factors, making it impossible to determine evaporation from air temperatures alone.

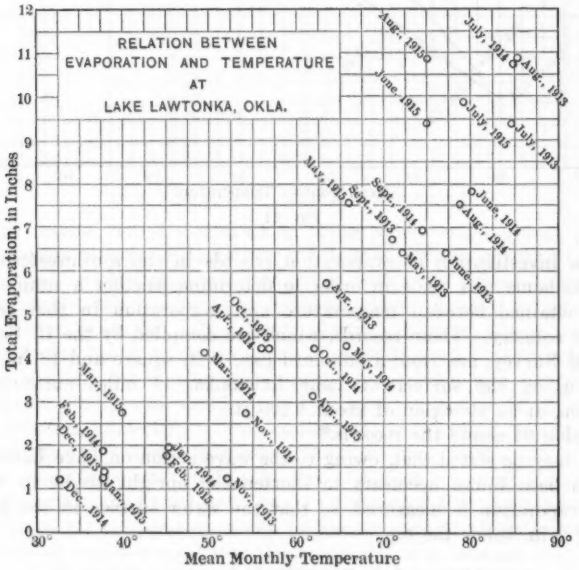


FIG. 22.

L. R. JORGENSEN,* M. AM. SOC. C. E. (by letter).—In this valuable paper the authors, in their estimate of evaporation depths from lake surfaces, have laid great stress on the influence of mean temperature and relative elevations. They mention other influences, but consider them of less importance. This reasoning no doubt is sound, and holds true for Lake Conchos and similar cases. Mr. Jorgensen.

The writer, however, desires to call attention to what in his judgment is a different case, and where the greatest stress will have to be laid on the influence of prevailing winds and the extreme dryness of the air.

It became of value to know the evaporation depths of Grant and Silver Lakes, in Mono County, Eastern California, in order to decide on the economy of developing both, or only one, of them, for storage purposes, and to find out how much water was available for irrigation purposes after allowance had been made for the losses. No evaporation data were on hand from the lakes themselves, but the yearly evaporation depths from a few points approximately 100 miles distant were known, and these data had to be interpolated to fit Silver Lake conditions.

The writer had been on the ground during the three hottest summer months, and during this time observed two things which were especially kept in mind when reasoning out the probable yearly evaporation depth from these lakes. These two things were the extreme dryness of the atmosphere, and the prevailing daily "trade" winds. Silver Lake is at Elevation 7 212, in a narrow canyon, from 1 500 to 3 000 ft. deep. There are extensive deserts about 8 miles north and east of the lake, and a few miles to the west the crest of the Sierra Nevada Mountains is always covered with snow. This condition creates the "trade" winds, the direction of which is down the canyon during the greater part of the day, and up the canyon after dark. This last condition may be due in part to the peculiar bend of Reversed Creek, from which it derives its name.

The rate of evaporation naturally depends on the ability of the air near the water surface to absorb water. The avidity with which air absorbs water increases as the humidity of the air decreases. If the humidity of the air is high, it will evaporate but little water, no matter what the temperature may be; and if the temperature of the humid air is much higher than that of the water, the water will soon cool the air in contact with it to such an extent that it can absorb no water at all, because it has reached its dewpoint at this new temperature. At such a stage the lake has been covered with an effective "vapor blanket".

The conditions at Silver Lake are such as to induce evaporation as much as possible. The humidity is very low, and therefore the air

* San Francisco, Cal.

Mr. Jorgensen.

in contact with the lake surface absorbs moisture quickly. Whatever vapor blanket tends to form is destroyed periodically by the wind, which brings new air in contact with the water surface. The temperature is fairly high during the summer; there is no rain during this time of maximum evaporation, and therefore this evaporation takes place every day. The two features at this place which tend to keep down evaporation are the high elevation, and the regular oval shape of the lake.

Through such reasoning, and from the data at hand,* it was estimated that the yearly gross evaporation from Silver Lake would be 4 ft. 6 in. This is probably high, considering the fact that the lake is frozen over in the winter, but, if so, it is on the safe side. It was also known that the irrigation duty of the water used in the valley below these lakes at practically the same elevation was 7 acre-ft. This high value is not due to heavy evaporation entirely, however, but partly to the porous character of the soil.

At Lake Conchos, fortunately, the rainy season coincides with the season of maximum temperature, and this condition undoubtedly tends to keep down evaporation loss; otherwise one would expect the yearly evaporation depths there to be higher than 55 in., due to the high average temperature and the very irregular shape of the lake.

Mr. Meyer.

ADOLPH F. MEYER,† M. AM. SOC. C. E. (by letter).—The writer has read this paper with much interest, and, in this discussion, does not desire to dispute the validity of the authors' principal conclusion, relating to the annual evaporation from Lake Conchos. He does, however, desire to present data which appear to disprove the subsidiary conclusion regarding the relation between evaporation depth and elevation above sea level. Inasmuch as the data on which the primary conclusion is based include Piche evaporimeter observations in Texas and New Mexico in 1887-88, the writer's discussion regarding the subsidiary conclusion also has some bearing, at least, on the method of arriving at the primary conclusion.

The authors state, regarding the relation between evaporation depth and elevation above sea level:

> "(*e*)—In the Great Plateau Region, and at elevations above sea level of 600 to 5 000 ft. and monthly mean temperatures higher than 70° Fahr., the increases in the evaporation depth at any given mean temperature are directly proportional to increases in elevation; but at monthly mean temperatures lower than 70° Fahr., and at the lower elevations, the evaporation depth at any given mean temperature increases with the increase in elevation, but less rapidly than in direct proportion."

* *Water Supply Paper No. 68*, and a pamphlet from the Truckee-Carson Experiment Farm.

† Minneapolis, Minn.

Mr. Meyer.

These conclusions are further presented in Diagrams (*a*) and (*b*) of Fig. 4. Although the line of zero evaporation is not given in Diagram (*a*), one is at once impressed with the fact that, according to this diagram, there would appear to be no evaporation on the Great Plateau at an elevation of about 3 000 to 4 000 ft. when the monthly mean temperature reached 30° Fahr., and at an elevation of about 2 000 to 2 500 ft., when the monthly mean temperature reached 40°, and that at elevations of less than 1 000 ft., there would appear to be no evaporation at monthly mean temperatures below about 45 degrees.

As to the applicability of the authors' conclusions regarding the relation of evaporation to elevation, the following statements appear in the paper:

"Also, it is believed that for places within the limits of the Great Plateau, from Mexico on the south to perhaps Colorado and Utah on the north, fair general approximations to local monthly and yearly evaporation depths may be read directly from Fig. 4 without other data than local mean temperatures and elevations.

* * * * * * *

"Since, at elevations above sea level, water boils at temperatures below 212° Fahr., it appears at least reasonable that at any given temperature water will vaporize or evaporate more quickly at high than at low elevations.

"On Diagram I of Fig. 13 are plotted the curves showing the relations between temperature and evaporation in 3-ft. square floating-pans at five measured evaporation stations on the Great Plateau. On each curve is marked the name and elevation of its station; and this diagram shows in a general way that at equal mean monthly temperatures the evaporation depth is greater at stations of higher elevation. The combined diagram of the Piche evaporimeter observations (Plate LXIII) also shows this in a general way.

* * * * * * *

"In the study made for Fig. 4, the values of Table 24 were extended upward to Elevation 7 000 ft., downward to 500 ft. below sea level (say as for Salton Sea and Dead Sea), and to the 2-in. increment of 16 to 18 in. monthly evaporation depth; and the values of Table 23 were extended to the same limits. This was done because it was surmised that the combined influence on evaporation depth of all other conditions except temperature and elevation might be relatively unimportant—perhaps so small as to permit the estimation of approximate evaporation losses directly from Fig. 4, not only for places on the Great Plateau, but even generally throughout the United States.

"However, the diagram of the extended Table 23 showed serious contradictions beyond 12 in. monthly evaporation (*e. g.*, at 400 ft. below sea level and 81.4° mean monthly temperature, the 6 and 18-in. lines intersect and give either 6 or 18 in. monthly evaporation), and hence it shows that values derived by extensions so far beyond the observed limits are very unreliable.

Mr. Meyer.

"Also, comparisons were made of the evaporation losses shown by such extended Fig. 4, with the observed evaporations at Salton Sea (263 ft. below sea level), Lake Tahoe (6 225 ft. elevation), and Coyote or Laguna Seca (240 ft. elevation)—all in California and outside the limits of the Great Plateau—and all such comparisons showed very serious disagreements, and indicated that Tables 23, 24, and 25, and Fig. 4 are not applicable with safety to places outside the limits of the Great Plateau."

In view of the fact that the authors believe that "for places within the limits of the Great Plateau, from Mexico on the south to perhaps Colorado and Utah on the north, fair general approximations to local monthly and yearly evaporation depths may be read directly from Fig. 4 without other data than local mean temperatures and elevations," notwithstanding the fact that extensions of Fig. 4 showed serious contradictions, and that comparisons of observed evaporation losses at Salton Sea, Lake Tahoe, and Laguna Seca, with values determined from such extended Fig. 4, showed very serious disagreements, and that elevation *per se* affects evaporation, "since, at elevations above sea level, water boils at temperatures below 212° Fahr.", further discussion of this subject would not appear to be amiss.

Piche Evaporimeter Observations.—In Table 17, and on Plate LXIII, the authors present Piche evaporimeter records for eleven stations in Texas and New Mexico; and, in referring to the Piche evaporimeter records which are presented as substantiating their subsidiary conclusion regarding the relation between evaporation and elevation, they state:

"These evaporimeter records* are stated to have been published in the *Monthly Weather Review* of the U. S. Signal Service, September, 1888, p. 235, and to have been records of observations made in the year, July, 1887–June, 1888."

Apparently, then, the authors did not examine the original source of these data.

Reference to the *Monthly Weather Review* of September, 1888, discloses the following facts, which are of interest in connection with the authors' use of the Piche records:

"The Piche evaporometer [evaporimeter] consists of a glass tube 9 inches long and 0.4 of an inch internal diameter. The top end is closed and has an eye for suspending the instrument. The tube is filled with water and on the open end a paper disk 1.2 of an inch in diameter is placed and held in position by a brass fixture sliding easily in a collar along the tube. Water is supplied to the paper from the tube by capillary action, and evaporates freely from it into the air, the amount depending on the dryness of the air and the velocity of the wind. There is a scale etched on the tube to cubic centimeters and tenths. The difference in the readings of the top of the column of

* "Public Water Supplies", Turneaure and Russell (1911), Second Edition, p. 60.

water gives the volume evaporated from the paper in the interval between the readings." Mr. Meyer.

These instruments were standardized so as to give comparable readings. It was found by experiment that the height of water in the tube made no appreciable difference in the rate of evaporation. It was found by Dr. Riegler, in 1879, that the thickness of the paper did not affect the evaporation more than might be expected from the difference in effective area, as represented by the relative areas of the edge of thin paper and of thick blotting paper. The same conclusion was reached by Professor Russell in his investigation.

According to this article in the *Monthly Weather Review*, which, by the way, was written by T. Russell, Assistant Professor, Signal Service:

"These instruments [referring to the Piches] were mounted in the thermometer shelters at the stations. They were read once a day at 10 P. M. during June, and at 8 P. M. during July, August, and September. The difference in the readings on successive days gave the amount of evaporation for the various days.

"The shelters on Signal Service stations are usually at a height of nine feet above the roof of building where situated. * * *

"To determine the relation between the evaporation from a water surface and that from the Piche, two Piches were compared, in the quiet air of a closed room, with two tin dish-evaporimeters, each 6.55 centimeters in diameter and 1.3 in depth. * * *

"The results of the experiments showed that the rate of evaporation from the dishes diminished regularly as its water surface fell below the edge of the vessel. The Piches evaporated 1.33 times that of the dishes, surface for surface, as compared with the dishes while quite full. The coefficient increased to 2.05 as the surface of the water in the dish fell.

"The coefficient used to reduce the Piche evaporations on stations to an equivalent water surface was 1.33."

Russell's Evaporation Table.—Russell's table giving "Depth of evaporation, in inches, at Signal Service stations, in thermometer shelters, computed from the means of the tri-daily determinations of dew-point and wet-bulb observations", first published in the September, 1888, *Monthly Weather Review*, and reprinted in Turneaure and Russell's book on "Public Water Supplies", and in a number of other publications, is based on the depth of evaporation, in inches, observed with Piche instruments, during June, 1888, at the stations shown in Table 65.

The evaporation at the 140 stations, given in Russell's table previously referred to, and from which the data used by Messrs. Duryea and Haehl were taken, does not represent Piche evaporimeter observations at the various stations for 12 months, but merely the computed evaporation determined by a formula derived from Piche observations for the single month of June, 1888, at the 18 stations listed in Table 65.

Mr. Meyer.

In deriving this formula, it was assumed, in the first place, that the evaporation observed with a Piche evaporimeter is 1.33 times that from a water surface. It was further assumed that, even though there was considerable variation in the observed wind velocity during June, 1888, and that, although at a wind velocity of 5 miles per hour, the evaporation from a Piche was found to be 2.2 times that from one in quiet air, at 10 miles 3.8 times, and at 15 miles 4.9 times, the effect of the wind could be neglected in the derivation of the formula. The wind velocity measured by an anemometer set up inside of the shelter at Washington City, for 8 days, gave a wind velocity of 3.48 miles an hour, as compared with a velocity of 6.63 miles outside, or only 52% of the velocity outside.

TABLE 65.

Station.	Temperature, in degrees Fahrenheit.	Evaporation depth, in inches.	Percentage of relative humidity.	Wind velocity, in miles per hour.	Barometer, in inches Hg.
Boston	66.8	5.16	65.0	10.2	29.8
New York	71.4	4.49	67.6	8.3	29.7
Washington	73.0	4.64	68.0	4.8	29.8
Cincinnati	74.2	6.22	56.6	6.1	29.3
Memphis	75.4	4.33	70.8	4.8	29.6
New Orleans	77.3	3.82	77.7	6.8	29.9
Chicago	67.4	5.59	64.1	10.3	29.2
St. Louis	73.2	6.18	68.5	9.7	29.3
Keeler	73.9	11.66	23.0	7.9	26.2
Yuma	85.6	13.87	25.3	7.3	29.6
El Paso	83.0	13.91	24.1	8.2	26.1
Dodge City	74.5	7.80	53.0	11.6	27.3
San Antonio	78.0	2.76	75.3	8.1	29.1
Omaha	70.0	7.01	63.2	7.6	28.7
Denver	68.4	9.42	31.4	8.0	24.7
St. Vincent	62.8	5.63	69.5	7.6	28.9
Helena	58.8	4.88	56.6	7.9	25.7
Boise City	64.2	5.83	48.8	3.1	27.0

In deriving the formula, the observed evaporation was first reduced on the basis of the observed barometric pressure, on the assumption that the evaporation varies inversely as the barometric pressure, that is, the observed evaporation at Denver, for example, where the barometer read 24.7 in., was assumed to be $\frac{30.0}{24.7}$, or 1.21 times the theoretical evaporation used in determining the other members of the equation. In other words, the tabular values for evaporation depth, based on the Piche measurements used by Messrs. Duryea and Haehl in Table 17, and on Plate LXIII, had already been corrected for elevation on the assumption that evaporation varies inversely as the barometric pressure.

Mr. Meyer.

From the observed evaporation for June, 1888, the following formula was derived by Russell:

$$\text{Evaporation (in inches per month)} = 30\left(\frac{A\,p_w + B\,(p_w - p_d)}{b}\right),$$

where

p_w = Vapor tension, in inches of mercury, corresponding to monthly mean wet-bulb temperature;

p_d = Vapor tension, in inches of mercury, corresponding to monthly mean dew-point temperature;

b = Mean barometric pressure, in inches of mercury;

$A = 1.96$;

and $B = 43.88$.

The values for A and B were derived from the observations for June, 1888, at the 18 stations listed in Table 65, by the method of least squares.

It is interesting to note the differences between the computed and observed evaporation for June, 1888, at two of the stations used by Messrs. Duryea and Haehl. At El Paso, Tex., the evaporation observed by the Piche evaporimeter, in June, 1888, was 13.91 in.; the computed evaporation was 13.6 in. At San Antonio the observed evaporation was 2.76 in.; the computed evaporation was 4.5 in.

In comparing evaporation from a Piche with evaporation from a reservoir, it is necessary to take account of the fact, stated by Russell in the *Monthly Weather Review*,* that:

"In the case of the Piche evaporometer the temperature of the evaporating water is strictly that of a wet-bulb thermometer exposed at the same place."

The Piche evaporimeter observations represent the evaporation at an average wind velocity, at the instrument, of approximately 3½ miles per hour. On the other hand, as stated by Russell:

"The effect of the high exposure of the shelters is to make the figures too great, the wind action being far greater at the height of the shelters than at the level of the ground. The evaporation taking place from a small paper disk, as in the results obtained with the Piche instrument, have a tendency to be too small, as the determining temperature of evaporation is that of a wet-bulb thermometer exposed under similar circumstances. In the case of a body of water the determining temperature of evaporation is nearly that of the average temperature of the air."

All the data used by Messrs. Duryea and Haehl on Plate LXIII are of no value whatever, in so far as efforts to show a relationship between evaporation and elevation are concerned, because, in the first place, they are based on an assumed variation of evaporation with

* September, 1888, p. 237.

Mr. Meyer.

elevation in the ratio of 30 divided by the barometric pressure, and, in the second place, all remaining differences in evaporation are directly attributable to changes in temperature and relative humidity. The evaporation at El Paso for June, 1888, is based on a relative humidity of practically 25%, and the evaporation at San Antonio on a relative humidity of practically 75 per cent. It must be apparent, then, that the computed evaporation at the 11 Weather Bureau stations, used by Messrs. Duryea and Haehl on Plate LXIII and in Table 17, cannot be used to determine the effect of elevation on evaporation, or even to substantiate a conclusion arrived at in another way.

Piche Evaporimeter Observations not in Agreement with the Authors' Deductions.—In order to show, further, that Russell's evaporation tables indicate no relationship between elevation and evaporation, and to show the differences in monthly evaporation as given by Russell and as determined from Fig. 4, a study has been made of Russell's values for the stations (situated on or near the Great Plateau) given in Table 66.

TABLE 66.

Station.	Approximate elevation above sea level, in feet.	Percentage of mean annual relative humidity (1887-1888).
Yuma, Ariz.	140	48
Keeler, Cal.	3 600	38
Winnemucca, Nev.	4 350	46*
Salt Lake City, Utah.	4 350	52
Denver, Colo.	5 300	52
Prescott, Ariz.	5 400	64

* The relative humidity at Winnemucca varied from 24.4% in August, 1887, to 83.4% in June, 1888. At all other stations the variation during the year was considerably less.

The monthly evaporation, according to Russell, at each of the stations listed in Table 66, is shown in Figs. 23, 24, and 25, together with a curve representing the average relationship between evaporation and temperature for each station. There is also shown, on each diagram, the relation between evaporation and temperature, as determined from Diagram (*a*) of Fig. 4, in order to indicate the lack of agreement between the conclusion reached by Messrs. Duryea and Haehl, with respect to the relation between evaporation and elevation, and Russell's evaporation values at other stations on or near the Great Plateau.

It will be noted that the writer has used a curve instead of a straight line for the purpose of better showing the relation between temperature and evaporation. In fact, this relation could be shown still better if separate curves were drawn for evaporation during the spring and during the fall, in accordance with the method adopted

Mr. Meyer.

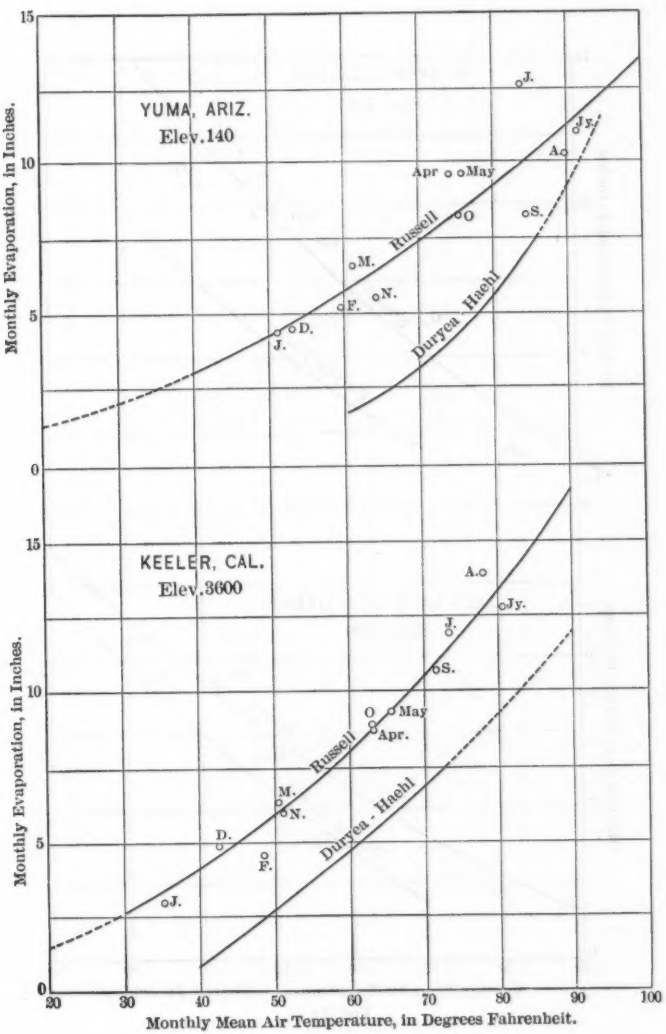


FIG. 23.

Mr. Meyer.

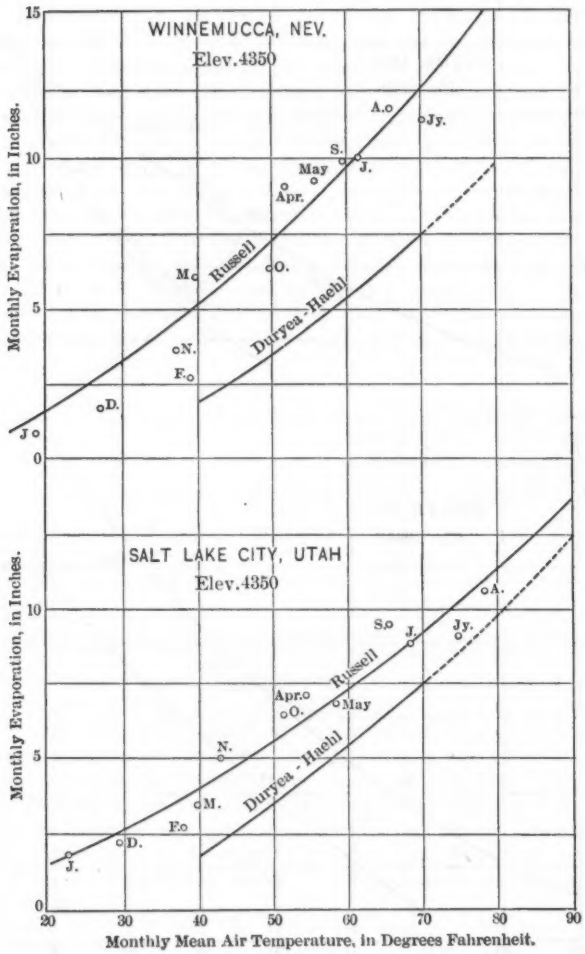


FIG. 24.

Mr. Meyer.

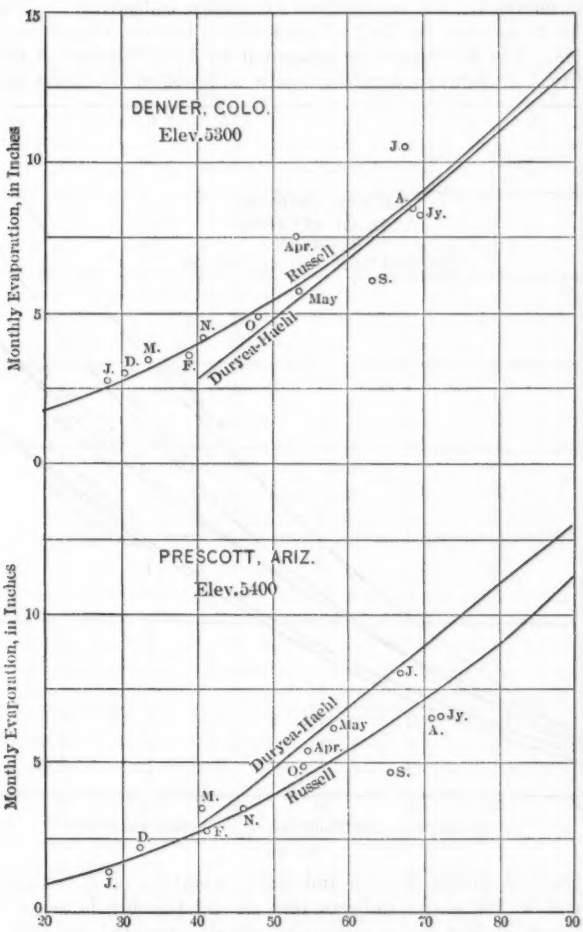


FIG. 25.

Mr. Meyer. by the writer in his paper "Computing Run-off from Rainfall and Other Physical Data".*

The curves for the six stations are shown collectively in Fig. 26, in order to indicate the lack of relationship between evaporation and elevation. The differences are accounted for by differences in relative humidity. If relative humidity were a function of elevation, the

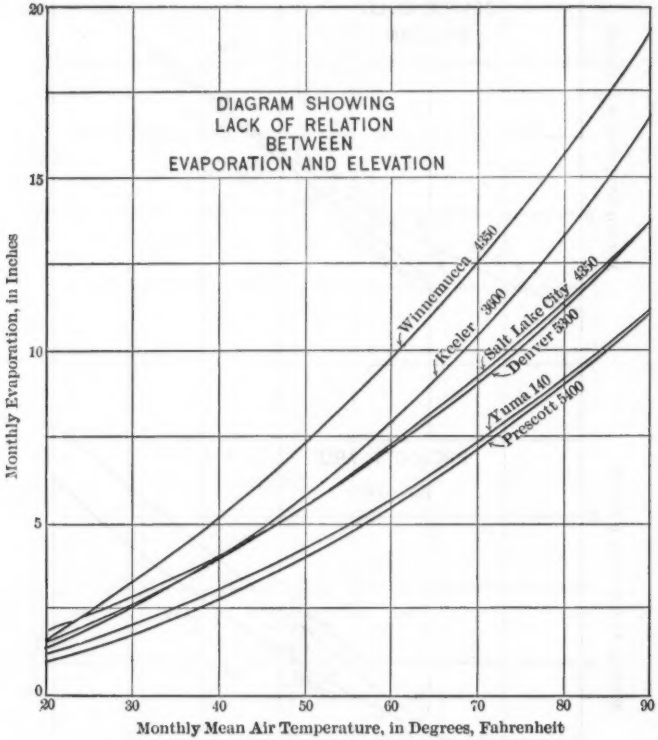


FIG. 26.

deductions of Messrs. Duryea and Haehl might be valid, but the data presented by the writer indicate that the relationship is merely incidental, and at best too uncertain to afford a rational basis for estimating evaporation.

If the authors' conclusions are limited in application to localities possessing similar climatic characteristics, then surely one would

* *Transactions*, Am. Soc. C. E., Vol. LXXIX, p. 1056.

look for at least some reference to climatic characteristics of the region under discussion, aside from mere records of temperature; that is, one would expect to find some record of relative humidity, wind velocity and direction, sunshine, etc. If the conclusions presented in Fig. 4 are to be of service at all, they must be applicable at least to points on the Great Plateau, as indicated by the title. The result of a comparison of the conclusions of Fig. 4 with Russell's evaporation tables fails to give confidence in the validity of those conclusions. Mr. Meyer.

Further data could readily be adduced from Russell's evaporation tables to show that, whatever effect elevation has on evaporation, it is merely incidental, and dependent on the reduction of relative humidity with increase in elevation. Even in the Great Plateau Region, however, there are many stations of low elevation at which the relative humidity is high, resulting in just the opposite relation between elevation and evaporation from that pointed out by Messrs. Duryea and Haehl. Pikes Peak, for example, has an elevation of more than 14 000 ft., yet the observed relative humidity is about 75%, as compared with a relative humidity of about 40% at Fort Grant, Ariz., with an elevation of 4 900 ft.

In passing, it may be remarked that Professor Bigelow's formula, quoted by Messrs. Duryea and Haehl, takes no cognizance of elevation as a factor in governing evaporation. Judging from extensive quotations from Professor Bigelow's writings, one is led to conclude that the authors have given weight to Bigelow's work; yet, on the other hand, it is difficult to see why they disregard completely the fact that Bigelow's formula does not contain elevation as a factor.

Relation between Evaporation and Elevation as Determined from Pan Measurements.—From the preceding discussion it is evident that the Piche evaporimeter records show no such relationship between elevation and evaporation as presented by the authors in Figs. 4 and 13.

As previously indicated, if relative humidity were a function of elevation, then elevation above sea level would be as good a measure of evaporation as relative humidity, but such a relationship between relative humidity and elevation does not exist, except as an accidental relation on portions of the Great Plateau, where regions of low precipitation are also regions of high elevation.

The foot-note to Diagram II of Fig. 13 indicates that the authors considered the records at El Paso, Tex., as unreliable, in so far as their use in determining the relation between elevation and evaporation was concerned, even though, in the discussion of the pan measurements, they state:

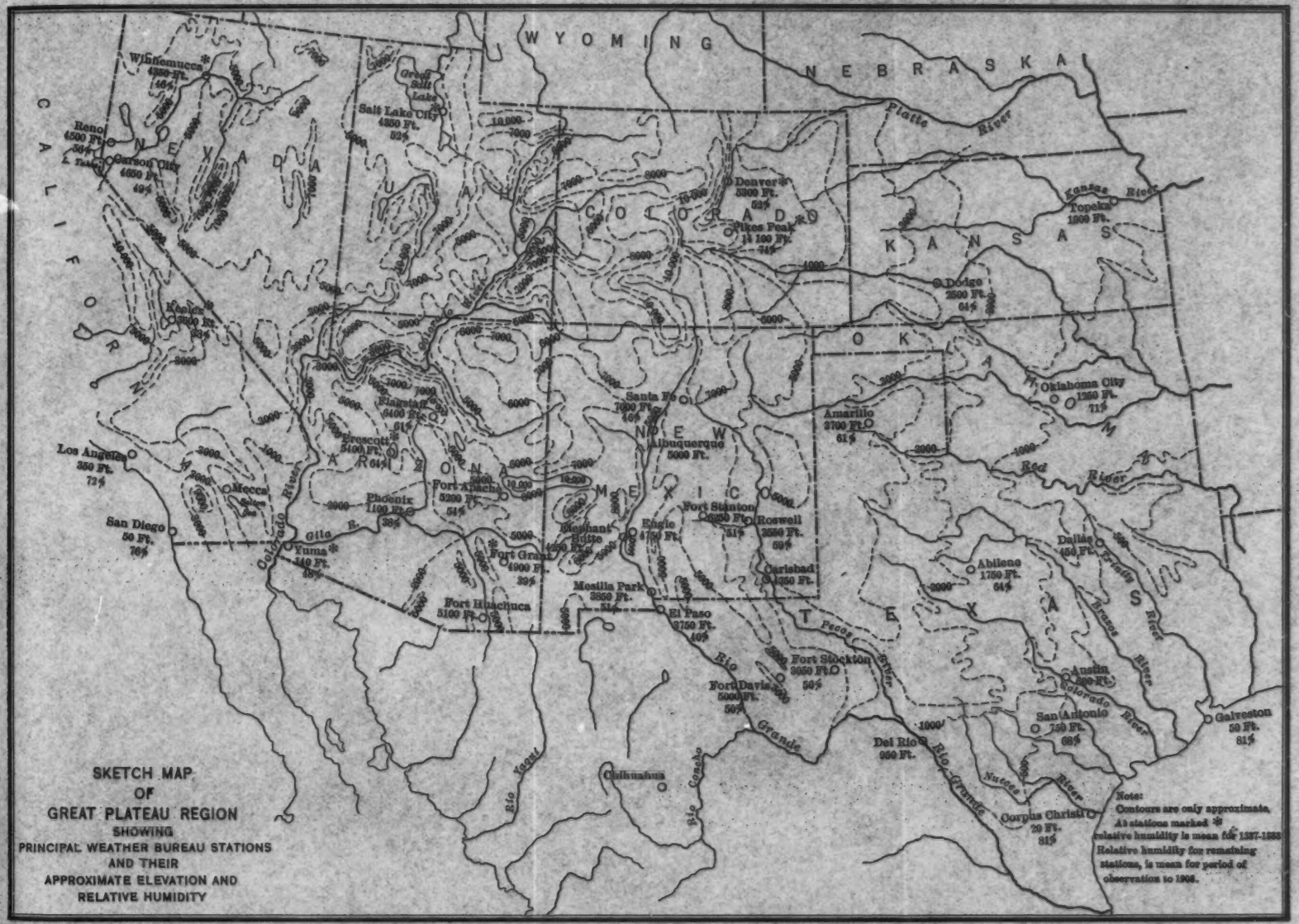
"Because of its comparative nearness, that at Elephant Butte is perhaps the most important (next to that at El Paso) of any of the Texas and New Mexico evaporation records available for the determination of the probable evaporation at Lake Conchos."

Mr. Meyer.

It is interesting to note, however, that the reason the El Paso records do not conform to the authors' deduction regarding the effect of elevation on evaporation is because of variations in relative humidity. This is shown by a comparison of the observed relative humidity at El Paso for the period covered by evaporation measurements, with the relative humidity observed at Santa Fé, N. Mex., a short distance north of Albuquerque. These records indicate that the average relative humidity during the summer months of 1889, 1890, and 1892, with temperatures of from 75 to 80°, is 31%, whereas the average relative humidity at Santa Fé, during the summer months of 1900 and 1903, for temperatures of from 75 to 80°, is 42 per cent. On the other hand, the relative humidity at El Paso, during November, December, January, and February, for temperatures of from 45 to 50°, is 45%, and the relative humidity at Santa Fé, during the winter months, when the temperature ranged between 40 and 50°, is 41 per cent. That is to say, the reason that the lines for El Paso and Albuquerque, in Diagram I of Fig. 13, cross each other is apparently because the relative humidity at high temperatures is less at El Paso, and consequently the evaporation greater, than at Albuquerque, and at low temperatures the relative humidity at El Paso is greater, and consequently the evaporation less than at Albuquerque. Moreover, the records indicate relatively uniform wind velocity at El Paso during the two periods under consideration, whereas those for Santa Fé indicate a somewhat higher wind velocity during the winter months. There appears to be good reason, then, for believing that the El Paso records do not conform to the authors' lines of Diagram II of Fig. 13 because the relative humidity at El Paso is exceptionally low, rather than because the evaporation observations at El Paso covered only portions of a month. Where deductions are properly made, it is believed that evaporation data covering 20 days are practically as good as those covering 30 days, although, when averaging results, less weight should be given to the average of 20 days' records than to that of a full month. An error sometimes made, however, and which is of consequence, is that of plotting the evaporation for 15 or 20 days out of a month against the monthly mean temperature, instead of against the mean temperature for the portion of the month over which the records extend.

It is interesting to note, further, that the general relationship pointed out by the authors in Fig. 13, between evaporation and the elevation of Austin, Carlsbad, Elephant Butte, Albuquerque, and El Paso, is to be expected from the differences in relative humidity at these or near-by stations. The relative humidity at Austin, Tex., should be about 68%, which is the relative humidity of San Antonio. A reasonable value for Carlsbad would appear to be 58%, on the

PLATE LXVIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXX, No. 1376.
MEYER ON
EVAPORATION FROM LAKE CONCHOS, MEXICO.



SKETCH MAP
OF
GREAT PLATEAU REGION
SHOWING
PRINCIPAL WEATHER BUREAU STATIONS
AND THEIR
APPROXIMATE ELEVATION AND
RELATIVE HUMIDITY

Note:
Contours are only approximate.
All stations marked * relative humidity is mean for 1887-1888
Relative humidity for remaining stations, is mean for period of observation to 1908.

basis of 56% at Fort Stockton and 59% at Roswell. Elephant Butte might be expected to have a relative humidity of about 50%, on the basis of 51% at Mesilla Park and 51% at Fort Stanton. The relative humidity for Albuquerque should be about 44%, on the basis of the records for Santa Fé during 1900 and 1903, which give 44% for that station. The relative humidity for El Paso, for the period covered by the records presented by the authors, was 36%, the normal relative humidity being 40 per cent. The location of these and other stations on or near the Great Plateau, is shown on the map, Plate LXVIII, which also gives very approximate contours for the region for which evaporation data are discussed in the paper. Mr. Meyer.

In conclusion, the writer desires to state that he believes records of relative humidity and wind velocity, combined with records of temperature, afford a far more accurate and reliable basis for estimating evaporation from reservoirs than Fig. 4. Sling psychrometer measurements of relative humidity are easily made. By combining pan evaporation observations with observations of temperature, relative humidity, and wind velocity for even 6 months, reasonably good estimates of evaporation from reservoir surfaces can be made by a proper analysis and comparison of these data with long-term records at near-by stations.

The monthly mean relative humidity, particularly at continental stations, as indicated in the writer's Fig. 4* of his paper "Computing Run-off from Rainfall and Other Physical Data", shows a quite definite variation from season to season. By comparison of observed values covering a portion of the year with similar values at a near-by long-term station, a reasonably accurate curve of variation in relative humidity for the whole year can be drawn. These values of relative humidity can then be used in connection with monthly mean temperatures and wind velocity, determined in a similar way, for the purpose of estimating monthly evaporation. This method of comparison between short-term and long-term records permits one to ascertain whether the year in which observations were made was an abnormal one, and thus avoid basing conclusions on the results of observations for a year, or a part year, which may represent either high, low, or intermediate values. Moreover, by this method of comparison, it is also possible to determine the evaporation which would result from a combination of meteorological phenomena favorable to very high evaporation losses. As the evaporation is almost certain to be high during a year of low stream flow, the necessity for considering exceptional rather than average values is apparent.

The writer desires to reiterate the statement previously made, that the criticism offered in this discussion does not apply to the authors'

* *Transactions*, Am. Soc. C. E., Vol. LXXIX, 1915, p. 1068.

Mr. Meyer. principal conclusion, but merely to their subsidiary conclusion relating to the effect of elevation on evaporation, and the suggested use of Fig. 4.

The intent of this discussion is not to find fault with the authors' analysis of the available data leading up to the estimate of the mean annual depth of evaporation from Lake Conchos, but rather to supplement their discussion of the relation between evaporation and elevation by pointing out that such a relationship is not fundamental, but merely incidental, and therefore true over such restricted areas as to make elevation an unsatisfactory and unreliable index of evaporation, even for large portions of the Great Plateau Region, and a totally inapplicable one elsewhere.

Mr. Hawgood. H. HAWGOOD,* M. AM. SOC. C. E. (by letter).—Messrs. Duryea and Haehl have rendered a service of great value in adding to the knowledge of a subject about which opinions are more common than recorded facts. Nothing could more strongly emphasize the present wide divergency of views than the statement made in the paper that two prominent engineers, judging of probable annual evaporation losses, gave opinions 151% apart. Nor are such divergencies of opinion by any means uncommon. Obviously, it is a field where there is need of a greater knowledge of facts. The paper is replete with information, and its conclusions are logical.

Measurement of evaporation from pans can be carried out with any desired degree of detail and accuracy. The difficulty lies in connecting such measurements with the actual evaporation from an expanse of water, such as a lake or reservoir. The opportunities for doing this are few, the difficulties many, and the results, by reason of the numerous uncertain elements, can never be considered as absolute, except perhaps in the case of comparatively small lined reservoirs. Seepage will always confuse the results. The perfectly watertight reservoir site is probably non-existent in Nature; otherwise springs would be rarities. In striking a balance between inflow and outgo, losses by inevitable seepage become charged to evaporation.

Again, the ratio of the area of the evaporating appliances to that of the reservoir is usually very small, too small for results to be accepted without some reservation. It is not to be expected that two or more pans, together presenting an area of a few square feet, could represent with precision areas frequently, as in the case of Lake Conchos, several million times greater, with all their variations of depths and exposures. The authors meet this question fairly by considering their results and conclusions as a reasonable basis for reliable estimations.

There is need of a series of comparative experiments with pans of the usual depth of 18 in. and others of several feet in depth, with

* Los Angeles, Cal.

Mr. Hawgood.

a view of determining the depth of pan associated with least evaporation, a condition which would most closely represent evaporation from a free water surface. The pan is supposed to represent the top of a column of water extending downward to the bottom of the reservoir, with all its fluctuations of temperature. Shallow pans probably give too high evaporative values.

The manner of construction and finish of the pans also needs investigation; that they exert a marked influence on evaporation has been known for a long time. It was the subject of experiments by Mr. Baldwin Latham some 34 years ago.* For the convenience of members, the following quotation is made, prefaced by the remark that Mr. Latham first used a pan supported by an outside contact ring, and that when the sun shone on the evaporator greater evaporation occurred by reason of the water creeping up the side of the vessel by capillary action, where it was speedily evaporated. He then substituted an evaporator supported by a floating ring separated from the vessel itself, the type used to-day.

"At once there was a large reduction in the amount of evaporation. The film of water now extended up the sides of the vessel both on the outside as well as the inside, and so they were kept cool. He, however, carried the experiment still further on a number of evaporators painted different colours. He observed that from a painted gauge, even if black, there was a less amount of evaporation than from a plain copper gauge. That was because capillary action was more energetic in the metal gauge than in the painted gauges. In order to test the matter, he took three evaporators made of copper, each 5 inches in diameter, and holding at least 1 foot depth of water. One of these he had slightly greased inside, and that and another copper evaporator were allowed to stand in a tank of water immersed to within 2½ inches of the top, while at the same time the third evaporator was freely exposed to the atmosphere. The evaporation from these gauges in the month of May, 1882, when compared with that of the floating gauge 12 inches in diameter, and a gauge painted white, but immersed in water were—

[12 in.] Floating Gauge.	[5 in.] Gauge in Air.	[5 in.] Gauge in Water.	[5 in.] Gauge greased in Water.	[5 in.] Gauge painted white in Water.
Inches.	Inches.	Inches.	Inches.	Inches.
3.665	6.355	5.495	4.085	4.175

"In the case of the gauge greased regularly, the coating was so slight that no oily matter ever appeared on the surface of the water. The size of the instrument had a marked influence on the amount of water evaporated, for the larger the vessel, in proportion to the area, the smaller was the marginal ring up which the water passed by capillarity, and from which it was evaporated."

That a little paint reduced the evaporation from 5.495 to 4.175 in., or 34%, is interesting and instructive.

* *Minutes of Proceedings*, Inst. C. E., Vol. LXXI, pp. 47-48.

Mr. Hawgood.

It does not appear that land pan observations can have any bearing on evaporation from open water. The water temperature and the vapor tensions at that temperature and at the temperature of the air are controlling elements in the depth of evaporation. The closer the water in the pan represents the temperature of the open water, the closer will the pan evaporation be to the open water evaporation. The water in a land pan partakes of the temperature of the surrounding ground, and can have no true relationship to the temperature of an adjacent body of water. The varying ratios between land pan and floating pan evaporations for different seasons of the year show the lack of influences in common, if anything is needed to show such a point.

If the pan is insulated from the earth, as is sometimes done, conditions are rendered still more dissimilar. On clear dry nights the insulated pan would sustain great loss of heat by radiation; it calls to mind the practice common in some parts of India and elsewhere of making ice by just such a process. The environments of any land pan are so void of parallelism with open water that the results they exhibit cannot be considered otherwise than equally at variance.

Mr. Comstock.

CHARLES W. COMSTOCK,* M. AM. SOC. C. E. (by letter).—The authors have contributed to the general fund of information on evaporation the results of 5 months' observations on two land pans and one floating pan, and about 1 month's observations on a second floating pan at Lake Conchos. Had they restricted themselves to legitimate conclusions from these observations, there would have been little to discuss except, perhaps, by those familiar with conditions in the vicinity of La Boquilla. They have gone into the field of speculation, however, and have built an unstable structure of generalizations on a very insecure foundation, *viz.*, one of the many Bigelow formulas. By this process, they have raised a doubt as to the validity of any of their conclusions.

Their criterion of correctness is apparent agreement with the Bigelow formula; therefore, discussion of that formula will not be out of place. Professor Bigelow has put forward a number of general formulas to connect evaporation with other meteorological conditions. The one used by the authors is that published in the *Monthly Weather Review*, for February, 1910.

Professor Bigelow's fundamental idea was that an evaporation formula containing, as a factor, the difference between the vapor tension of saturation at the temperature of the water surface and that at the dew point of the air, cannot be satisfactory because it does not agree in form with the rational formula derived by the integration of the differential equation for diffusion in one dimension in a quiet

* Denver, Colo.

Mr. Comstock.

atmosphere. As a matter of fact, we know that diffusion in a quiet atmosphere can take place only under the most carefully arranged and controlled laboratory conditions, and never in any outdoor surroundings. However, he adopted a form in which the ratio of vapor tensions, instead of their difference, is the controlling factor. There are many objections to this. As the vapor tension at the dew point enters in the denominator of the fraction, the rate of evaporation into perfectly dry air would be infinite. The absurdity of this is apparent. It is true that perfectly dry air is not found in Nature, but such a condition could be easily provided in the laboratory. Professor Bigelow, thus far, has totally ignored this consequence. Again, even if the air became saturated, evaporation would not cease, according to the Bigelow formula, as no factor therein becomes zero. Professor Bigelow, in commenting* on this feature, says:

"It is, however, probable that even when $e_s = e_d$, and the vapor pressure in the air is the same as that at the water surface, there is a flow of vapor in the tubes required to maintain the vapor pressure e_d near the water."

This explains nothing. If the air is saturated, there is no room for more water vapor. If the saturated air is partly or wholly removed by the wind, and its place is taken by unsaturated air, the flow of vapor from the water surface can go on. This, no doubt, is what Professor Bigelow had in mind, but it does not cover the assumed case of equal vapor pressures in the air and at the water surface. By his formula, water must go on evaporating in a closed chamber filled with saturated air, though it is not apparent what is to become of the vapor.

There is a still more important inconsistency in Professor Bigelow's argument. He rejects the idea that rate of evaporation depends on difference of vapor tensions at water temperature and dew-point temperature, but he determines the vapor tension at the dew point by the use of the psychrometer, notwithstanding that the psychrometric tables are calculated from a formula derived by consideration of the very relation which he discards.† If this relation is not true, the psychrometric tables do not give the temperature or vapor tension at the dew point, and the constancy of coefficients, on which Professor Bigelow relies to prove the correctness of his formula, is entirely misleading.

The authors' Subsidiary Conclusions (*a*), (*b*), and (*c*) are those of Professor Bigelow, and, in fact, are based on mere repetition of his figures and argument. Although it is not believed to be important, it is not clear why the authors should persist in squaring Professor Bigelow's round pans. The writer has been unable to find

* *Monthly Weather Review*, 1908, p. 439.

† Hann, "Lehrbuch der Meteorologie," Second Edition, 1906, p. 164.

Mr. Comstock.

anything in any account of the Salton Sea campaign, or of the preliminary campaign at Reno, which justifies the assumption of square pans, and the use of round pans at Reno is clearly shown by the halftone engravings.*

Subsidiary Conclusion (*a*) deals with relative depths of evaporation from pans of different sizes. Here the authors quote the values of the coefficient in the Bigelow formula, as given in "Abstract of Data, No. 4," issued by the United States Weather Bureau.† Professor Bigelow gives somewhat different values for this coefficient.‡ They are compared in Table 67.

TABLE 67.

Diameter of pan, in feet.	Coefficient in Bigelow Formula:	
	"Abstract of Data, No. 4."	*Monthly Weather Review.*
2	0.042	0.040
3	.....	0.042
4	0.036	0.037
6	0.031	0.029
12	.....	0.025
Lake surface.	0.023 to 0.025	.....

Whichever set is accepted, the values of the coefficient are regarded as measuring the relative depths of evaporation from pans of the corresponding diameters. Professor Bigelow states:‡

"The evidence is decisive, * * * that water evaporates much faster from small pans than from large pans. This is because the wind action tends to clear a small pan of vapor, making over it a dryer mixture, than it does a large pan or water surface, where vapor is merely transported from one side of the water area to the other side of it. Especially, over a lake or reservoir the vapor is carried along from point to point without drying the mixture of vapor and air resting on the water surface."

If it is admitted as a fact that water has been observed to evaporate faster from small pans than from large ones, it is too great a tax on our credulity to accept the explanation here offered as accounting for differences between pans 2, 3, and 4 ft. in diameter. That any atmospheric movement which can be called wind would clear water vapor from a 2-ft. pan more quickly or more completely than from one 3 or 4 ft. in diameter, is inconceivable.

* Fig. 3 on p. 25, and Figs. 5*a*, 5*b*, 5*c*, and 5*d*, on p. 27, *Monthly Weather Review*, February, 1908.

† Published subsequently in *Engineering News*, Vol. 63.

‡ *Monthly Weather Review*, July, 1910, p. 1133.

Mr. Comstock.

Professor Bigelow says,* further:

"In order to test the ratio of evaporation from pans of different sizes, our records include the following combinations: (1) A 4-foot pan self-registered hourly and a 2-foot pan alongside on the ground near Tower No. 1; (2) a row of 3 pans, 2-foot, 4-foot, 6-foot in diameter, on a platform on Tower No. 3, about half a mile from shore, and as near the water as was practicable; (3) a row of 4 pans, 2-foot, 4-foot, 6-foot, 12-foot, on a series of adjoining rafts floating in the Salt Creek slue in calm water. The ratios are quite steady and the results have been incorporated into the final value of the coefficient."

For the first two of the combinations here described, he does not give the detailed results of the observations or the coefficients calculated for the different pans. For the third combination, he gives the following values of the coefficient:

2-ft.	pan	0.031
4- "	"	0.030
6- "	"	0.026
12- "	"	0.025

The observations at Indio, in 1907, by Dr. C. Abbe, Jr., showed very clearly that the greater depths of evaporation from the smaller than from the larger pans in that instance were due to higher temperatures of the water in the smaller pans. Such temperature differences are to be expected in land pans, but cannot exist in floating pans. The Indio observations certainly contain nothing to support the hypothesis that the size of the pan *per se* has any influence on the depth of evaporation under otherwise identical conditions, or to justify the remarkable suggestion that such observed differences are to be accounted for by the action of the wind in clearing away the vapor more quickly from a 2-ft. than from a 4-ft. pan.

The ease with which Professor Bigelow reaches "final conclusions" and the readiness and frequency with which he changes them, are illustrated by comparing his statement of "decisive evidence" in July, 1910, with his equally dogmatic assertion of 1908: "The size of the pans makes an insignificant difference in the amount of evaporation, when they are under identical local conditions."†

This was written after the completion of the Reno campaign and the analysis of its results.

In the absence of more information than has been vouchsafed thus far, the writer may perhaps be pardoned if he fails to see the "decisive" evidence relative to the influence of size of pan.

Subsidiary Conclusion (*b*) has to do with relative evaporation depths from land pans and floating pans. The discussion of this subject by the authors is complicated by their use of the assumed ratios

* *Monthly Weather Review*, July, 1910, p. 1134.

† *Ibid*, February, 1908, p. 38.

Mr. Comstock.

for pans of different sizes under identical conditions. It seems certain that water will evaporate more rapidly from land pans than from floating pans under otherwise similar conditions, because the vapor tension at the dew point will be lower over the land pans, and because, as a rule, the temperature of the water in land pans will be higher, especially during the day, than in floating pans. Whether the authors' ratio of 80% is justified, however, is another question. Their observations at Laguna Seca seem to support this conclusion, but their work at Lake Conchos leads to a ratio approximating 85%, as do also the Granite Reef observations. The Salton Sea figures and those obtained at California, Ohio, should be rejected because they involve the at least uncertain ratios assumed to exist between evaporation depths from pans of different diameters. If the results at Coyote, Granite Reef, and Lake Conchos (5 months) are combined, the mean is found to be 83.4%, and this seems to the writer more nearly justified than the 80% adopted by the authors. If the separate ratios are weighted by the lengths of periods of observation, the mean will be about 82 per cent.

With respect to Subsidiary Conclusion (*c*), the authors have again followed Professor Bigelow, and their conclusion seems to be based rather on his dicta than on any numerical data. The quotation credited to Professor Bigelow, beginning on page 1855, is from the report of the Conference Board appointed to outline the Salton Sea campaign, and was signed by Professor Bigelow, Mr. G. K. Gilbert, of the United States Geological Survey, and C. E. Grunsky, M. Am. Soc. C. E. It was written in advance of any work at Salton Sea or at Reno, and was purely speculative. Even as an hypothesis, it is so far at variance with well-known facts as to warrant its summary rejection. The quotation is in part as follows:

> "As the air moves across a reservoir and gradually becomes charged with moisture, its rate of absorption diminishes, and the average rate of evaporation for a broad surface is therefore less than for small surfaces."

Does any one think of wind as a mass of air, maintaining its form and the identity of its particles, moving like a rigid body horizontally across a water surface for 10 or 15 miles, and absorbing moisture as it goes? The most casual observation of a floating flag, a column of smoke, or the falling snow shows how far such a concept is removed from the fact. The air particles move with velocities which change rapidly, even abruptly, in magnitude and direction, not only with the position of the point of observation, but also at any chosen point. The motions have vertical components, frequently of high velocity. The atmosphere is cut in all directions by surfaces of discontinuity, and the whole swirling, tumultuous mass is so thoroughly mixed that

it is impossible to imagine the existence for more than a few minutes at a time of a stratum differing materially in its content of water vapor or in any other respect from the average of the whole. Mr. Comstock.

Nor does the wind velocity, as observed by an ordinary anemometer of the type used by the United States Weather Bureau, give any measure of this mixing effect. These anemometers are affected only by the horizontal component of motion, and they add such components numerically, irrespective of direction. Furthermore, they are arranged so as to record the passage of each mile (or each kilometer) of wind, regardless of whether the motion is uniform for the entire time, or fitful and variable in the extreme.

The most graphic description of the condition which the writer is here attempting to depict is the following by the late Professor S. P. Langley:*

"A prominent feature * * * is that the higher the absolute velocity of the wind, the greater the relative fluctuations which occur in it. In a high wind the air moves in a tumultuous mass, the velocity being at one moment perhaps 40 miles an hour, then diminishing to an almost instantaneous calm, and then resuming.

"The fact that an absolute local calm can momentarily occur during the prevalence of a high wind, was vividly impressed upon me during the observations of February 4, when chancing to look up to the light anemometer, which was revolving so rapidly that the cups were not separately distinguishable, I saw them completely stop for an instant, and then resume their previous high speed of rotation, the whole within the fraction of a second.

* * * * * * *

"* * * The velocity, which was, at the beginning of the interval considered, nearly 23 miles an hour, fell during the course of the first mile to a little over 20 miles an hour. This is the ordinary anemometric record of the wind at such elevations as this (47 metres) above the earth's surface, where it is free from the immediate vicinity of disturbing irregularities, and where it is popularly supposed to move with occasional variation in direction, as the weather-cock indeed indicates, but with such nearly uniform movement that its rate of advance is, during any such brief time as two or three minutes, under ordinary circumstances, approximately uniform. This then may be called the 'wind,' that is, the conventional 'wind' of treatises upon aërodynamics, where its aspect as a practically continuous flow is alone considered. When, however, we turn to the record made with the specially light anemometer, at every second, of this same wind, we find an entirely different state of things. The wind starting with the velocity of 23 miles an hour, at 12 hrs. 10 mins. 18 secs., rose within 10 seconds to a velocity of 33 miles an hour, and within 10 seconds more fell to its initial speed. It then rose within 30 seconds to a velocity of 36 miles an hour, and so on, with alternate risings and fallings, at one time actually stopping; and, as the reader may

* "The Internal Work of the Wind," 1893, pp. 9 *et seq.*

Mr. Comstock.

easily observe, passing through 18 notable maxima and as many notable minima, the average interval from a maximum to a minimum being a little over 10 seconds, and the average change of velocity in this time being about 10 miles an hour.

* * * * * * *

"In order to insure clearness of perception, the reader will bear in mind that the diagram does not represent the velocities which obtained coincidentally, along the length of two miles of wind represented, nor the changes in velocity experienced by a single moving particle during the given interval, but that it is a picture of the velocities which were in this wind at the successive instants of its passing the fixed anemometer, which velocities, indeed, were probably nearly the same for a few seconds before and after registry, but which incessantly passed into, and were replaced by others, in a continuous flow of change. But although the observations do not show the actual changes of velocity which any given particle experiences in any assigned interval, these fluctuations cannot be materially different in character from those which are observed at a fixed point, and are shown in the diagram. It may perhaps still further aid us in fixing our ideas, to consider two material particles as starting at the same time over this two-mile course: the one moving with the uniform velocity of 22.6 miles an hour (33 feet per second), which is the average velocity of this wind as observed for the interval between 12 hrs. 10 mins. 18 secs., and 12 hrs. 15 mins. 45 secs., on February 4; the other, during the same interval, having the continuously changing velocities actually indicated by the light anemometer * * *. Their positions at any time may, if desired, be conveniently represented in a diagram, where the abscissa of any point represents the elapsed time in seconds, and the ordinates show the distance, in feet, of the material particle from the starting-point. The path of the first particle will thus be represented by a straight line, while the path of the second particle will be an irregularly curved line, at one time above, and at another time below, the mean straight line just described, but terminating in coincidence with it at the end of the interval. If, now, all the particles in the two miles of wind were simultaneously accelerated and retarded in the same way as this second particle, that is, if the wind were an inelastic fluid, and moved like a solid cylinder, the velocities recorded by the anemometer would be identical with those that obtained along the whole region specified. But the actual circumstances must evidently be far different from this, since the air is an elastic and nearly perfect fluid, subject to condensation and rarefaction. Hence the successive velocities of any given particle (which are in reality the result of incessant changes in all directions), must be conceived as evanescent, taking on something like the sequence recorded by these curves, a very brief time before this air reached the anemometer, and losing it as soon after."

In such wind, as distinguished from the wind of popular imagination, Professor Bigelow's "vapor blanket" must needs have the tenacity and water-proofing qualities of the best rubber in order to remain intact and afford the protection with which it has been credited.

Mr. Comstock.

The authors quote from Professor Bigelow as follows:

"* * * in the arid regions of the West it seems probable that a lake or reservoir evaporates about five-eighths as fast as an isolated pan placed outside the vapor blanket; in other words, this vapor blanket seems to conserve about three-eighths of the water that would otherwise be lost by the evaporation."*

Let us see on what evidence the existence of this "vapor blanket" rests, and what justification there is for the statement that it conserves three-eighths of the water which would otherwise be lost.

At the end of the Reno campaign, Professor Bigelow worked up his results in great detail on the basis of a formula, which he afterward discarded, and with certain assumptions in respect to which he has since changed his mind—notably the influence of size of pan. Having computed the values of the coefficient in his assumed formula for pans exposed at different heights above the surface of the ground, he then "adjusted" them by a perfectly arbitrary process, without a shadow of rational justification, and thus arrived at a limiting value, of which he says:

"The maximum value of C is 0.0430, *which corresponds to a maximum evaporation outside the vapor blanket covering the reservoir,* and it is at some asymptotic distance."

The italics are Professor Bigelow's. The ratios of the values of C at the water surface to this asymptotic value vary from 0.395 to 0.72 for the different towers, and on this evidence is based the statement that the "vapor blanket" conserves three-eighths of the water which would otherwise evaporate.

The reasoning is illogical, and the whole process is unscientific and arbitrary. The only conclusion which can properly be drawn from Professor Bigelow's involved calculations is that water evaporates less rapidly near the ground than it does at some place where the wind velocity is greater and the vapor pressure in the air less, and this was common knowledge before the inauguration of the Reno campaign.

Again, the authors quote from "Abstract of Data, No. 4" as follows:

"These data indicate a nearly uniform rate of evaporation over the area of the water, beginning a short distance from the shore."

The data referred to are observations at two points, 7 000 ft. apart, the most distant less than 1½ miles from shore. The Salton Sea was 45 miles long, from 10 to 15 miles wide, and covered an area of 440 sq. miles. What kind of logic is this? If we are to leap such enormous gaps from a few observations to a broad generalization, almost anything may be proved inductively.

* *Monthly Weather Review*, February, 1908, p. 39.

Mr. Comstock.

It is practically certain that conditions affecting evaporation are not uniform over an area of 440 sq. miles, at least for any length of time. If they are not, the measured evaporations and the coefficients calculated from them would differ from point to point, and Professor Bigelow's accepted ratio of evaporation from a large water surface to that from a floating pan is left without any foundation.

Compare the minute and detailed measurements and calculations which were carried out in the evaporation observations at Reno and Salton Sea, with the off-hand way in which other data are assumed. For example:*

"The automatic record indicates that the sea has fallen about 4.60 ft. annually, or, allowing for the inflow, we have a little more or less than 6.0 ft. by evaporation. It is probable that the inflow from the New and Alamo rivers, together with the precipitation flowing in from the surrounding country amounts to something like 1.5 ft."

And again, from the same source:

"If 70 ins. is admitted as the amount evaporated from the Salton Sea, there remains 38 ins. as the difference between the water in the sea and that in the lower swinging pan."

What is the basis for the assumed inflow? The only channels bringing water to the sea, on which any attempt at measurement has been made, are the Alamo and New Rivers. Gauging stations were established on these streams on June 24th, 1909. The station on the Alamo was discontinued on January 6th, 1912. Concerning that station, the description is, in part:†

"Conditions for obtaining accurate discharge data are poor. The channel is constantly scouring or filling as the stage fluctuates."

In respect to the New River station, the report is even less favorable.‡

"Conditions for obtaining accurate discharge data are exceedingly poor. The great amount of fine silt carried by this stream causes continual changes in the channel. The current is light at low stages. Floods occur at long intervals and are extremely torrential.

"Conditions at this station during 1909 were fairly good up to the middle of August, when heavy rains fell in the Imperial Valley and surrounding country. A considerable flood occurred on New River, washing out the earth approaches to the bridge, and changing the channel so completely that measurements made prior to August are not comparable with those that will be made later. Probably the channel was fairly stable after October 1, 1909, but sufficient discharge measurements have not been made to define the new rating curve. Estimates of flow are therefore withheld."

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* "Abstract of Data, No. 4."

† *Water Supply Paper No. 300*, U. S. Geol. Survey, p. 411.

‡ *Ibid*, p. 415.

Mr. Comstock.

The last reported measurement in 1909 was made on July 30th, and the only one since that date was on February 4th, 1910.

The information obtained from such gauging stations, although no doubt the best that circumstances afforded, is certainly not sufficiently reliable to justify its use in the calculation of coefficients in a formula intended for general application.

The rainfall and run-off data are even less reliable. Referring to the Imperial Valley, F. T. Robson, Assoc. M. Am. Soc. C. E., says:*

"In such desert country, where the rainfall is scant, that which does occur is frequently in intense storms or in local cloudbursts. Monthly records, therefore, are of little value as indicating the probable run-off, and each storm at each observation station must be considered, instead of the monthly totals."

Under such conditions, any estimate of the total precipitation on a drainage area of 7 020 sq. miles is subject to enormous errors, as the areas covered by these very intense storms cannot be known, and the resulting run-off is so uncertain that it cannot even be guessed at intelligently.

Thus, in the attempt to determine the evaporation from Salton Sea, there is just one element which permits of exact measurement, *viz.*, the fall in the water surface, and this, it may be assumed, is accurately known. The inflow through two channels has been only crudely measured, and the rainfall and resulting run-off have been little better than guessed. Yet, after 3 years of elaborate measurements, refined analysis, and hair-splitting numerical computations, Professor Bigelow uses such figures as a basis for the ratio which is the sole practical result of his investigation, and the authors quote it to the tenth of 1 per cent.! The involved and complicated discussion in which the Reno and Salton Sea observations have been buried has given a false idea of their precision and of the value of the conclusions drawn from them. We must not be misled by mathematical intricacy masquerading as profound scientific analysis.

Referring now to Mr. Robson's computations from the Salton Sea data, which the authors have cited, it seems to the writer that an incorrect use of some of the figures has been made. Mr. Robson adds the observed rises in the surface of the sea and assumes that the total represents the discharge from the Alamo and New Rivers. This does not seem to be warranted. It is conceivable that the discharge from these streams, together with inflow from other sources, might be just sufficient to maintain the water level constant, in which case, by Mr. Robson's method, the discharge of the rivers would be nothing. His estimate of accretions to the sea from rainfall, though no doubt the best that could be made from the existing information, is too uncertain

* *Transactions*, Am. Soc. C. E., Vol. LXXVI, p. 1521.

Mr. Comstock.

to admit of its use in the determination of the evaporation depth from Salton Sea.

As a result of a careful examination of all the data and arguments, the writer feels justified in saying that nothing has yet been adduced to show that the evaporation from a lake surface is less than from a pan floating in the lake (except for possible difference in temperature of the water inside and outside the pan), much less to establish the ratio of evaporation depths to the third decimal place.

It has not even been shown that there is commonly a difference of temperature between the water in a floating pan and the water in which it floats, and the writer believes that such difference, if it exists at all, will be insignificant, unless the pan is very shallow. A. A. Weiland, M. Am. Soc. C. E., now State Engineer of Colorado, maintained continuous evaporation observations for 1½ years, in 1908-09, on two reservoirs near Pueblo, Colo. He used cubical pans, 3 by 3 by 3 ft., floated in the reservoirs. Concerning these observations, he writes, in a personal letter, as follows:

"For some time we took both the temperature of the water in the pan and in the reservoir, but this was abandoned on account of the two readings being practically the same."

We come now to Subsidiary Conclusion (*d*): Here the authors apparently have allowed their wish to find a simple relation between evaporation depth and some meteorological element commonly of record, to outweigh their scientific judgment. At the end of their discussion of the relation between evaporation depth and monthly mean temperature they say, on page 1877:

"* * * the conclusion seems warranted that (at least for the Great Plateau and for the complete cycle, averaging a year) the monthly depth of evaporation increases with the mean monthly temperature, usually by an approximate 'straight-line' relation."

The writer contends that this conclusion is not justified by the authors' own diagrams, even if their premises are accepted, which in itself is impossible.

They cite six stations in Texas and New Mexico, at which measurements of evaporation from pans have been made, and they say concerning them:

"A study of these data * * * develops the fact that for each station there exists a stable average relation between its monthly depth of evaporation and its monthly mean temperature; * * *

"It is well known that many other conditions besides temperature affect the depth of evaporation, such as prevalence and velocity of the wind, humidity, vapor tension, barometric conditions, etc. However, these other conditions are almost entirely unknown in most cases

Mr. Comstock.

where engineers are called on to estimate the probable annual evaporation—whereas mean temperatures frequently are of record. It is fortunate, therefore, that the joint effect of all the climatic conditions except mean temperature is so small (either because of the relative unimportance of each, or because some tend to counteract others) that a direct comparison of mean temperature with evaporation depth at any station on the Great Plateau leads to an average relation so stable and so essentially accurate that its use is justifiable in estimating evaporation depths, even when unaided by a knowledge of the other climatic conditions which affect evaporation."

These statements are of the sort generally classified as "important if true". If they are to be accepted, the departures of observed evaporation depths from the curves drawn on the diagrams must be regarded as accidental errors, and are subject to discussion as such. The writer, therefore, has prepared from each of the diagrams, El Paso, Albuquerque, Carlsbad, and Elephant Butte, a table showing the departure of each observed evaporation from the corresponding curve, and from these has computed the probable error of a single observation at each station. The evaporation depths were scaled from the diagrams.

At El Paso the mean departure (disregarding sign) of the observation from the curve is 1.05 in., or 15% of the mean monthly evaporation. The probable error of any single observation is 0.89 in.

At Albuquerque the mean departure is 1.107 in.—again 15% of the observed mean monthly evaporation—and the probable error of a single monthly observation is 1.04 in.

At Carlsbad the mean departure is 0.53 in.—13% of the mean monthly evaporation—and the probable error is 0.52 in.

At Elephant Butte the mean departure is 0.90 in.—12% of the mean monthly evaporation—and the probable error is 0.80 in.

Similar computations for Austin and Lake Avalon were not made because of the small number of observed values at those stations.

Such probable errors certainly do not warrant the statement:

"* * * that a direct comparison of mean temperature with evaporation depth at any station on the Great Plateau leads to an average relation so stable and so essentially accurate that its use is justifiable in estimating evaporation depths, even when unaided by a knowledge of the other climatic conditions which affect evaporation."

On the contrary, they point unmistakably to the omission of one or more elements having a direct causal relation with evaporation.

Does it not seem strange, also, that the temperature-evaporation relation should be a straight line at Lake Avalon, and a curve of higher degree at Carlsbad, only 5 miles distant? That out of six stations examined, straight lines are considered satisfactory relations at four, and that other curves must be used at two?

Mr. Comstock.

Up to this point the authors have adhered rigidly to Professor Bigelow's formula and to every deduction he has made from it. In their attempt to establish a simple relation between monthly mean temperature and evaporation depth, they have entirely ignored, not only the Bigelow formula, but all formulas heretofore suggested. Their assumption, that evaporation depth at any station may be expressed as a function of monthly mean temperature only, leads inevitably to the conclusion that vapor tension of saturation, vapor tension at dew point, and the wind term, are functions of the air temperature at the station, and of no other variables. That any combination of these elements, by the Bigelow or by the Dalton formula, integrated over a period of one month, should reduce to a constant multiplied by the monthly mean temperature is not only inconceivable in theory, but is not indicated by the authors' own facts.

The very large probable errors resulting from their diagrams have already been mentioned. In addition, it should be pointed out, as the authors have recognized, that if the linear relation between temperature and evaporation obtains, the cumulative diagrams on Plate LXV should be straight lines. As a matter of fact, these curves depart very markedly from straight lines, and all in the same way. The authors say of this:

"It is noticeable that all the curves of Plate LXV * * * are of a similar 'ogee' form at all the measured stations, and for a whole year do not depart greatly from a straight line."

It is, of course, possible to draw through any group of plotted points a straight line which will represent the law of their relation better than any other straight line, but this does not justify the choice of the straight line in preference to any other curve. It would seem that the striking similarity of all these mass-curves and their departure from a straight line should have suggested to the authors that their assumed temperature-evaporation relation was incorrect, rather than appealing to them as evidence in its favor.

Turn now to the Piche evaporimeter records, diagrams of which are published on Plate LXIII, and which the authors regard as confirmatory of their assumption: Of them the authors say:

"These evaporimeter diagrams show clearly (as do those of the pan-evaporations at the six stations) that the monthly depth of evaporation varies with the monthly mean temperature, in general in accordance with a 'straight-line' relationship."

A glance at the diagrams will show that, for six of the eleven stations, namely, Rio Grande City, Abilene, El Paso, Fort Davis, Fort Stanton, and Santa Fé, curves similar in character to those used by the authors for the Austin and Carlsbad records would fit the plotted

points much more closely, and that, in these instances at least, the "straight-line relationship" is forced to an absurd degree. Mr. Comstock.

In reference to this matter, it is worth while to look into the history of the Piche evaporimeter and the observations made with it, to which the authors have referred. It has frequently been said that the indications of the instrument are equivalent to pan measurements, but no one has cited an ultimate authority for the statement.

The Piche instrument is one of the numerous devices, intended to provide a ready and easily portable means for the measurement of evaporation, which were put forward from time to time before the complex nature of the problem was generally recognized. It was first described under the name *"atmismomètre".** It has since been subjected to numerous tests in comparison with other devices for the same purpose, and with erratic and uncertain results. Mr. William Napier Shaw has stated† that there were three difficulties with the Piche instrument: (1) A certain difference of pressure between the air inside and outside the tube is required to force the bubbles through the paper, and this may not be constant; (2) the condensation of water on the sides of the tube at the water surface; (3) the readings are sensibly affected by variations in temperature and barometric pressure.

In 1879, Riegler found the evaporation from a Piche evaporimeter to be 2.03 times that from a free water surface. In October and November of the same year, Kunze found values of 1.05, 1.12, and 1.09 for this ratio; in April, 1880, the same experimenter found 1.49, and in July, 1.37. In 1890, Houdaille concluded:

"Daily evaporation, as measured by the Piche atmometer, is very irregular as compared with that from an evaporating surface more naturally exposed to the action of the wind and nearer the temperature of the air."

Describing the work with these instruments by Professor Thomas Russell, of the United States Signal Service, in 1888, Professor Cleveland Abbe wrote:‡

"Comparing equal surfaces of paper and water the Piches evaporated 1.33 times as much as the dishes when the dishes were full, but the ratio increased steadily up to 2.05 in proportion as the surface of the water in the dishes fell lower and lower.

* * * * * * *

"* * * The different Piche evaporometers agreed with each other more closely than the different dishes of water. The height of water in the tube of the Piche evaporometer or its pressure on the disk of paper made no appreciable difference in the evaporation. These comparisons were all made with apparatus installed in thermometer shelters,

* In Bulletin de l'Association Scientifique de France, in 1872.

† Quarterly Weather Report, Meteorological Office (London), 1877.

‡ *Monthly Weather Review*, June, 1905, p. 254.

Mr. Comstock.

i. e., cubical boxes, three feet on a side, with open louvers, they therefore correspond to a very gentle movement of the wind over surfaces that are protected from the direct rays of the sun.

"An anemometer set up inside the shelter gave velocities only one-half those recorded on the outside. It was found impossible to make any satisfactory allowance for the wind as recorded in the standard location of the anemometer. The average velocity of the wind for a year at all Signal Service stations was estimated to amount to about 8.5 miles per hour, and the general observations of evaporation must be considered as holding good for that velocity in the free air or one-half of that velocity in the shelter.

"The effect of the wind on evaporation was determined by direct experiment, using the 28-foot whirling machine set up in the interior of the Pension Office Building, with which anemometers had been standardized. One Piche evaporometer was suspended in the adjoining quiet air, the other fixed to the outer end of the long arm of the whirling machine and driven at a given rate of motion for half an hour, when both instruments were observed by very accurate weighing instead of the less accurate scale reading. Then the two Piche evaporometers were interchanged and another similar set of observations made. The evaporation from the moving Piche was greater than that from the quiet one by the factors in the following table, which holds good for an average temperature of 83.7° F. and a relative humidity of 50 per cent.:

Velocity. Miles per hour.	Factor.
0	1.0
5	2.2
10	3.8
15	4.9
20	5.7
25	6.1
30	6.3

"If barometric pressure is lowered the evaporation is increased, but no opportunity was offered for repeating these whirling experiments at very low pressures and they may be assumed to hold good for a barometric reading of 30.00 inches. On the other hand the evaporation computed for 30.00 inches was reduced theoretically by Professor Russell for the effect of the low pressure at any given high station by using the ratio 30.00 inches divided by the mean pressure at the station as the barometric reducing factor.

"As the amount of evaporation increases with the temperature and dryness of the air Professor Russell also reduced the observations at Washington to a theoretical value at any other station by using the additional factor

$$A\ p_w + B\ (p_w - p_d)$$

where p_w is the vapor tension corresponding to the wet-bulb thermometer, and p_d is the vapor tension corresponding to the dew-point. Using the monthly means of observations at the eighteen stations occupied by the Signal Service the arbitrary coefficients A and B were deter-

mined so as to satisfy the whole range of observations, and Professor Russell found $A = 1.96$, $B = 43.9$. Mr. Comstock.

"It will be seen, therefore, that by means of the coefficient 1.33 Professor Russell passed from the Piche to the evaporation from the surface of water in dishes in shelters. By means of the tabular results of experiments with the whirling machine he passed from the wind in the shelter to the natural wind outdoors. By means of the barometric coefficient $\frac{30}{b}$ he passed from the lower to the higher altitudes. By the coefficients A and B he passed from the temperature and relative humidity prevailing in the shelter at Washington to the conditions prevailing at any point in the United States. Of course, however, this whole process was a series of approximations, and although it is the best that has yet been done, still it is in great need of revision.

"The amount of evaporation from natural surfaces exposed to full sunshine can not be estimated even roughly from the records of observations made within instrument shelters. The latter may apply fairly well to forest areas, cloudy weather, and shaded mountain sides, but work in direct sunshine is needed in order to supplement the work done by Professor Russell. Such measurements under natural conditions have indeed been recorded by several investigators, notably Stolling, at St. Petersburg, FitzGerald, at Boston, the Army Engineers, at Milwaukee, Wis., and more recently Carpenter, at Fort Collins, Colo. With the first of these results Professor Russell compares his own work and shows that if we assume a velocity of 3.5 miles per hour within the shelter at Boston, or 7.1 miles outside the shelter, then the computed evaporation, 34.40 inches, agrees closely with the observed evaporation, 39.11, for one year. For New York the computed evaporation is 40.60 inches, the observed, at the Croton Water-works, 39.21 inches.

"For Milwaukee during three years, 1862, 1863, and 1864, but only during the warmer months of each year, observations were made by the Chief of Engineers by weighing a large shallow vessel filled with water set up inside a thermometer shelter. The mean of these three years, for the months of June to September, gave a total for these four months of 20.60 inches, the corresponding total computed by Professor Russell is 14.70, a large difference for which no explanation is offered. For the corresponding months at Chicago, in 1888, the Piche evaporometer gave 23.87 inches, and the theoretical formula for normal atmospheric conditions gave 19.60 inches.

"In both cases it is likely that the normal conditions assumed by the formula differed largely from the actual conditions of the years in question. In general, therefore, although Professor Russell's formula will hold good for the moist climate of the eastern United States it may give too small values for the hot, dry climate of the interior. He estimated its accuracy as correct within 20 per cent., but added that it must be remembered that the formula and the figures given by him for all Signal Service stations represent only the possibilities of evaporation under normal conditions, and not the actual evaporation occurring from natural surfaces."

Mr. Comstock.

From the experiments and formula just described, and by the use of psychrometer readings at various stations, have been computed the tables of evaporation generally published in textbooks, *e. g.*, "Public Water-Supplies", by Turneaure and Russell; "Irrigation Engineering", by H. M. Wilson. In the former work the table is headed "Calculated Monthly Evaporation", but in the latter the impression is given that the tabular figures are the results of direct observations with the Piche instrument. Unfortunately, many people know of these tables who know nothing of the method of their construction, and, therefore, they accept them as the last word on the subject.

Professor Bigelow says:*

"We used six improved Piche evaporimeters at Reno, and compared them directly with the evaporation from the pans at the same elevations and under like conditions, with the hope of substituting these more sensitive instruments for the large pans. * * * At Reno the evaporation was rapid enough to exhaust the water in about three hours in strong winds during the afternoon, so that the rate of the fall of the water was about 55 times greater in the tube than in the open pans, an advantage in accurate reading, *provided the ratio is a constant.*"

The italics are his. He then presents a table of values of this ratio, the range being from 41.7 to 76.5, with a mean for all of 56.2. Obviously, the indications of the instrument are worthless for the purpose of measuring evaporation.

Professor Bigelow then computes "that the evaporation averages 1.177 times faster from each cm^2 of paper to each cm^2 of the free water surface". He concludes: "It was a disappointment to find that this promising piece of apparatus must receive further study and development."

From the foregoing account of the history of and experience with the Piche evaporimeter, it is clear that its records, on which the authors rely so confidently to support their assumed evaporation-temperature relation, are unworthy of serious consideration. Even if they were dependable, six of the eleven stations used by the authors distinctly negative the straight-line assumption.

Having shown that at four of the six pan-evaporation stations used by the authors the assumption of evaporation as a function of mean monthly temperature only, leads to absurdly high probable errors, and that the Piche evaporimeter records are worthless and do not support the assumption, even if they were not, the writer believes he is justified in discarding Subsidiary Conclusion (*d*).

The authors' discussion of Subsidiary Conclusion (*e*), namely, relation between evaporation depth and elevation above sea, is clouded

* *Monthly Weather Review*, February, 1908, p. 38.

by the intimate admixture of the assumed evaporation-temperature relation. Here, again, they ignore Professor Bigelow, who wrote:* Mr. Comstock.

"There is no evidence that water in pans of the same size evaporates faster on the Plateau levels, 4,000 feet, than at sea level, and the formula should not contain any barometric pressure term."

In this respect they were wise. Professor Bigelow's failure to find a barometric term is not surprising, in view of his obscure and doubtful reasoning, and his conclusion on this point is at variance with experience and experiment.

In 1789, H. B. de Saussure made comparative observations on the Col du Géant, where the barometric pressure was 18 inches 9 lines, and at Geneva, Switzerland, where the barometer stood at 27 inches 3 lines, from which he concluded that "other things being equal, a lowering of the pressure of the air by approximately a third makes the quantity of evaporation more than twice as great".

Numerous laboratory experiments have shown beyond question that the air opposes a mechanical obstacle to the escape of liquid particles in the form of vapor, and that this obstacle is greater at high than at low pressures. It has been usual to introduce the barometric pressure in such a way as to make the rate of evaporation inversely proportional to the pressure. This is arbitrary, and, although perhaps sufficient to cover all natural conditions, is certainly not correct, as it would result in an infinite rate of evaporation in a vacuum, no matter what the temperature. However, if it is even approximately correct, the relation sought by the authors between evaporation and altitude above sea level must be very complex, because it contains implicitly the relation between altitude and barometric pressure.

The authors have not established any explicit relation between altitude (or barometric pressure) and evaporation, but have put forward a double relation involving altitude, monthly mean temperature, and evaporation. Since, for reasons already given, the temperature-evaporation relation must be rejected, the altitude-temperature-evaporation relation falls with it.

The net results of the paper, in respect to the subsidiary conclusions, may be summarized as follows:

(*a*).—Relative evaporation depths from pans of different sizes.—No relation established; and no differences shown to exist except when the water temperatures differ.

(*b*).—Relative evaporation depths from land-pans and floating-pans.—Very few acceptable data submitted; and these indicate a ratio of from 82 to 85 per cent.

(*c*).—Relative evaporation from reservoir surfaces and pans floating thereon.—It has not been shown that there is any difference, nor has any good reason been advanced why there should be.

* *Monthly Weather Review*, July, 1910, p. 1133.

Mr. Comstock.

(*d*).—Relation between evaporation depth and mean temperature.—None has been shown, and it is practically certain that none exists unless other meteorological conditions are included simultaneously.

(*e*).—Relation between evaporation depth and elevation above sea.—Same comment as on (*d*).

We come now to the authors' Principal Conclusion, namely, the yearly depth of evaporation from Lake Conchos.

They measured the evaporation from one floating pan for an aggregate of 112 days distributed through 6 months, and from another floating pan for 10 days in all, distributed through 3 months. These pans were about 2 miles apart, and both were within 2½ miles of the eastern end of a reservoir which, when filled, will be 25 miles long and will cover about 60 sq. miles, and which, at the time of the observations, covered 16 sq. miles. After ultra-refined speculations as to the differences in evaporation from 2, 3, and 4-ft. pans, and insisting on distinguishing between square and round pans used for such measurements, they calculated the annual evaporation over the whole reservoir from a few days observations in one corner of it.

From the observations on floating pan No. 2 (112 days) the authors estimate the total evaporation for 6 months, January to June, inclusive, to be 48.31 in. Using their estimated percentage, 50.4%, for these months, the annual evaporation is found to be 95.85 in.

The writer has calculated the percentage distribution of evaporation at thirty stations in the United States, and has taken the mean for each month. The result shows 46% of the annual evaporation for the months, January to June, inclusive. If this figure is used, the annual evaporation at Lake Conchos, calculated from pan No. 2, is 105.02 in.

The authors' estimated evaporation from measurements on floating-pan No. 4 (10 days observations) is 15.50 in. for the months, January to March, inclusive. Their estimated percentage for these months is 17.8%, giving an annual evaporation depth of 87.08 in. The writer's percentage for these months is 13%, giving 119.23 in. annual evaporation.

From these quantities, which are the only real data in the case, the annual evaporation loss is 91.46 in., using the authors' percentages, or 112.12 in. if the writer's percentages are correct. Not one sound reason for reducing the evaporation depth below at least the lower of these figures, has been advanced.

It is not improbable that the rate of evaporation differed to an important degree from point to point on the lake, but the authors' observations did not show this, and, in the absence of information on this point, they should have accepted the results of their measurements instead of seeking to reduce them, on entirely unjustified grounds, to conform to a preconceived notion.

Mr. Comstock.

It seems hardly probable that the various elements involved in a determination of evaporation by balancing inflow and outflow can be known with sufficient accuracy to warrant the use of this method. There must have been much uncertainty as to the volume stored and as to rainfall and run-off. For 4 months—October to January—the estimated precipitation on the lake surface was more than 2 in., and for 7 months—October to April—more than 3 in. These figures are based on measurements at two stations 25 miles apart; and, in a region of torrential storms, the mean of two stations cannot be safely taken as the average precipitation over a considerable area, nor can the local run-off be ignored.

Even if the authors' method is followed, there is some doubt as to the correctness of the results which they have reached. Their percentage of annual evaporation for October to January, inclusive, is 21.7%, and for October to April, inclusive, 44.1 per cent. The corresponding figures from the writer's calculations are 18 and 37 per cent. These give 66.89 in. evaporation from the 4 months' observations, and 68.65 in. from the 7 months' measurements, or a mean of 67.82 in. This is nearly one-fifth more than the authors have found, and is believed to be based on percentages more probable than they have used.

The writer believes that all evaporation formulas thus far suggested are erroneous as to form, because of the manner in which the wind term is introduced. The Dalton formula is the basis for all except Professor Bigelow's various forms, and these have no rational justification. In the Dalton and allied formulas the rate of evaporation is made to depend chiefly on the factor $(e_s - e_d)$, where e_s is the vapor tension of saturation at the temperature of the water surface, and e_d is the vapor tension at the dew point of the air. There is every reason to believe that this form is correct. The effect of the wind is to clear away the saturated or nearly saturated air, leaving in its place drier air, that is, air in which e_d is smaller. The wind term, then, should be introduced by making e_d a function of wind velocity, air temperature, and psychrometer difference, and e_s a function of water temperature only. In such a formula it is believed that the apparent anomalies in evaporation calculations will disappear.

It is useless to urge that a formula must contain only those elements that are commonly of record. If we are to calculate evaporation, or anything else, we must take into account all the elements which influence the effect in question. If we do otherwise, we mislead ourselves by a pseudo-precision which reminds one of the man who paced the diameter of a circle and computed the area to five decimal places.

It would be extremely convenient if evaporation depended only on monthly mean temperature, but that does not justify such an assumption.

Mr. Grunsky.

C. E. GRUNSKY,* M. AM. SOC. C. E. (by letter).—The writer desires to express his appreciation of the attempt made by the authors of this paper to establish the relation between mean monthly temperature and the evaporation from an open water area. He, too, has had occasion to study this problem, and believes that if a dependable relation can be pointed out it will prove of no small service to the engineer.

While Assistant State Engineer of California, the writer was placed in charge of evaporation studies at Kingsburg, Cal., the results of which have been published.† The writer also made use of these studies in a discussion of evaporation from the Salton Sea which he published‡ in 1908.

In 1906, together with Dr. G. K. Gilbert and Professor F. H. Bigelow, he was a member of a Conference Board charged with outlining a programme for the evaporation work subsequently carried out at the Salton Sea and at Reno by the U. S. Weather Bureau under the supervision of Professor Bigelow.

The writer urged Professor Bigelow to devote some time and attention to the evaporation problem from the standpoint of the engineer, in order that a fairly reliable foundation might be laid for determining from known weather conditions, temperature, humidity, wind movement, and rainfall, with due regard to altitudes and location, the monthly and annual evaporation rate. The published results of Professor Bigelow's observations will bear witness to the fact that the writer's appeal was of no effect, and that the engineer cannot get from those results the required basis for making, from weather conditions alone, a fair approximation of evaporation.

Messrs. Duryea and Haehl assume that Professor Bigelow used square pans in the evaporation work done by him at Salton Sea. In reading the papers by Professor Bigelow,§ the writer could find nothing to indicate that the pans used there and elsewhere in his evaporation studies were square. The photographs of the work at Reno show that circular pans were used there, and the description thereof confirms this. Professor Bigelow, in describing the pans used at Reno,‖ refers to them as circular, 2 and 6 ft. in diameter and 10 in. deep. He says, further, that the depth of water in the pans was measured by a glass tube,

"graduated to cubic centimeters, the scale being nearly in millimeters, and the tube being drawn to a narrow neck at the bottom. A plunger consisting of a plug on a copper wire was fitted to move

* San Francisco, Cal.

† "Physical Data and Statistics", William Hammond Hall, M. Am. Soc. C. E., State Engineer of California, 1886, p. 379; and again published by the writer in *Bulletin No. 100*, U. S. Dept. of Agriculture, p. 323.

‡ *Engineering News*, Vol. 60, August 13th, 1908, pp. 163–166.

§ *Monthly Weather Review*, 1908 and 1910.

‖ *Ibid*, February, 1908.

Mr. Grunsky.

up and down with the finger as desired, * * *. Raising this to the level of the eye the top of the meniscus of water was easily read off to a fraction of a millimeter on the vertical scale. No account was taken of the volume, but the scale was read from time to time at the same point in each pan, and the difference of height between reading measured the amount lost by the evaporation."

In the last paper by Professor Bigelow,* he refers to the "diameter" of the pans which he used. He would not have made the mistake of referring to a diameter in describing a square pan. The pans at the Salton Sea, as well as at Reno, were circular. Professor Bigelow says:

"In order to test the ratio of evaporation from pans of different sizes, our records include the following combinations: (1) A 4-foot pan self-registered hourly and a 2-foot pan alongside on the ground near Tower No. 1; (2) a row of 3 pans, 2-foot, 4-foot, 6-foot in diameter, on a platform on Tower No. 3, about half a mile from shore, and as near the water as was practicable; (3) a row of 4 pans, 2-foot, 4-foot, 6-foot, 12-foot, on a series of adjoining rafts floating in the Salt Creek Slue in calm water. The ratios are quite steady and the results have been incorporated into the final value of the coefficient, $C_2 = 0.023\ (1.23)^n$ for 4 hour intervals,

where $n = 0$ for large open water areas,
$n = 1$ for 6-foot pans,
$n = 2$ for 4-foot pans,
$n = 3$ for 2-foot pans,
$n = 4$ for ordinary dry air.

The value of the coefficient for $n = 1$ is fairly well determined, and it is interpolated for $n = 4$. These should be further verified if possible."

It will be noted that the pans were circular, and that those at the water surface were not "floating", in the ordinary sense of the term.

On inquiry at the U. S. Weather Bureau, it has been learned from Mr. B. C. Kadel, who was in actual charge of the field work under Professor Bigelow, that the pans used were round (as will be seen from the reproduction of the photograph, Fig. 27) and that the rafts were arranged in line off shore with a dummy raft next to the shore. The pans were supported so that they were partly immersed in water

"but the water inside the pans was kept at a higher level than the water outside in order that the pressure would always be in the same direction. The 12-ft. pan was so large that it was almost impossible to buoy it much higher than the water in the bay."

In reference to the same pans, Mr. Bernard R. Laskowski, of the U. S. Weather Bureau at Denver, who assisted in the work, says: "The pans that were on rafts were balanced so that they rested about 3 or 4 in. in the water."

* *Monthly Weather Review*, July, 1910, pp. 1133–35.

Mr. Grunsky.

The results, relating to ratios of evaporation rates from various-sized pans, which were determined by Professor Bigelow, apply to land pans on platforms and pans generally with water surface above that of any surrounding water, and not to floating pans or those embedded in the soil. Furthermore, these comparisons apply to a factor in a formula, and not to the actual evaporation, and are therefore only approximations to the ratios of actual evaporation.

It may be added that the principal cause of the varying rates of evaporation from pans of different diameters is probably the acceleration of evaporation by capillary action at the sides of the pans. Under the cloudless sky of the Salton Sea region, where the Bigelow experiments were made, the metal rims of the pans above the water became very hot, with undoubtedly an appreciable effect on evaporation. The ratio of perimeter to area increases as the area of the water surface in the pans increases. This suggests that, perhaps, for all practical purposes, the formula for the Bigelow coefficient might be written

$$C_2 = 0.023 + 0.010\, c\, \frac{p}{a},$$

where p is the wetted perimeter of the pan, a is the area of the pan, and c is a coefficient to be determined for various materials of which pans are constructed; for various climatic conditions; and for the position and possibly, too, the depth of the pan. The coefficients determined for pans placed on platforms or freely swinging, as at Salton Sea, or for pans partly immersed, should not be applied to those embedded in the soil, nor to those floating in the water (with water at the same level as the surrounding water), the sides of which are cooled in large measure from the outside, and not entirely by the evaporation of the water film attracted by capillary force on the inside.

For 10-in. deep land pans not embedded in the soil, in a climate comparable with that at Salton Sea, the value of c is unity, and the formula may be written

$$C_2 = 0.023 + 0.010\, \frac{p}{a}.$$

This would make, for land pans in a Salton Sea climate:

$C_2 = 0.043$ for pans 2 ft. in diameter,
$C_2 = 0.036$ " " 3 " " "
$C_2 = 0.033$ " " 4 " " "
$C_2 = 0.030$ " " 6 " " "
$C_2 = 0.023$ for large water areas.

These values are in fairly close agreement with those recommended by Professor Bigelow.

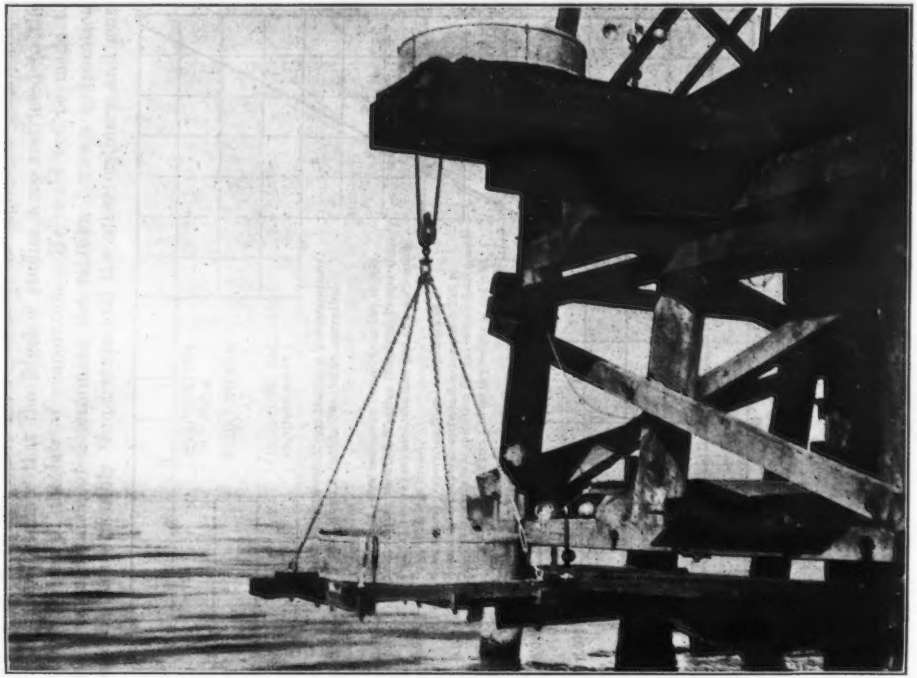

FIG. 27.—TOWER NO. 2, PANS 1 AND 2, SALTON SEA EVAPORATION CAMPAIGN.

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Further observation, it is confidently expected, will show that, for floating pans, the value of c is small, and that the size of the pan has much less effect on the evaporation rate than observed from pans above the water at Salton Sea.

It is unfortunate that the Bigelow studies were restricted to the determination of the law of evaporation. He could not be made to see the importance of determining the relation between evaporation and the mean monthly temperature and the other ordinary and gen-

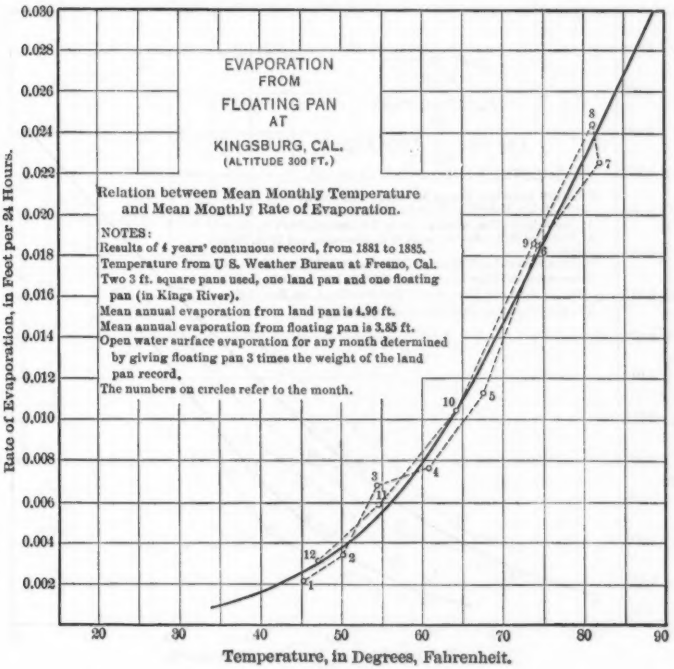

FIG. 28.

erally readily ascertainable weather conditions, such as wind movement and relative humidity. On this subject the writer took occasion to communicate with Professor Bigelow, both before and after he made his first tentative suggestion of a formula,* but without success. His attention was called to the fact that the mean water surface temperature of any inland lake or reservoir will bear some more or less definite relation to the mean temperature of the air, if this mean tem-

* *Monthly Weather Review*, February, 1908.

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perature is determined for sufficiently long periods, as, for example, a month. It is possible that periods of 10 or 20 days would have an advantage over a 30-day basic period, but the convenience of using the month as the time unit, owing to the availability of climatic records arranged by months, is too apparent to need discussion. The writer brought to Professor Bigelow's notice an article in which the preponderating influence of temperature on evaporation had been pointed out and in which the use of mean monthly temperatures as a guide

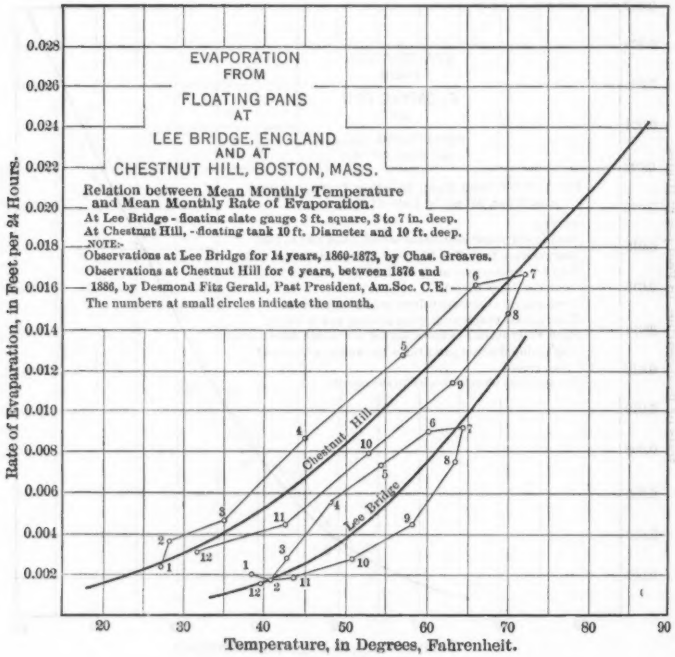

FIG. 29.

in approximating the evaporation from a water surface such as that at Salton Sea was illustrated. This article* was prepared by the writer in order to show that, throughout regions in which climatic conditions are similar, it may be assumed that errors due to more or less wind movement in one locality when compared with another, and those due to small differences in relative humidity and other similar causes, may be neglected, and, for all preliminary engineering purposes, evapora-

* *Engineering News*, Vol. 60, August 13th, 1908, p. 163.

tion from an open water surface may be determined from mean monthly temperature. Mr. Grunsky.

It will be apparent, however, that this can only be done after the relation between the mean monthly temperature and the rate of evaporation at one or more selected points during the calendar months has been determined experimentally. In the article referred to, the evaporation observations made at Lee Bridge, near London, during the 14 years, 1860-1873, by Mr. Charles Greaves,* the 6-year studies by

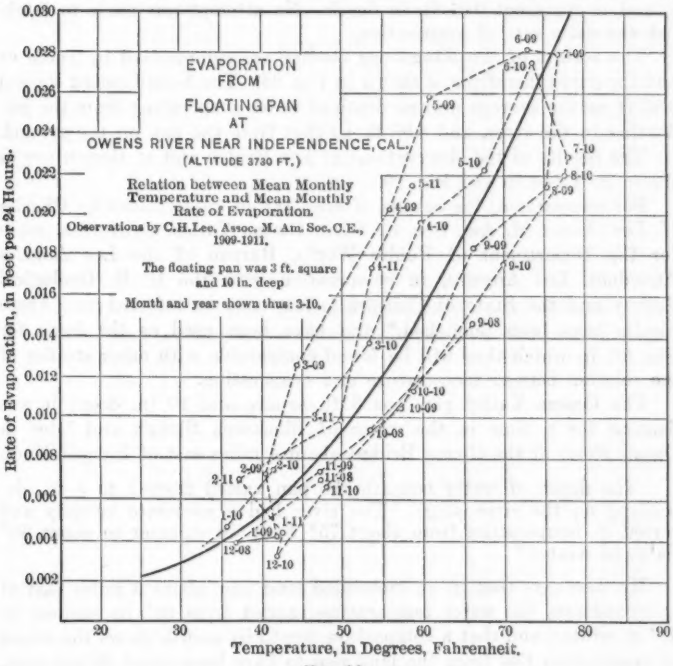

FIG. 30.

Desmond FitzGerald, Past-President, Am. Soc. C. E., 1876 to 1886,† at the Chestnut Hill Reservoir, Boston, and the 4-year records by the writer‡ under direction of William Hammond Hall, M. Am. Soc. C. E., State Engineer of California, at Kingsburg, Cal., are noted, and the relation between evaporation and the mean monthly temperature, as approximated from the Kingsburg record, is indicated.

* *Minutes of Proceedings*, Inst. C. E., Vol. XLV, 1875–76, p. 19.
† *Transactions*, Am. Soc. C. E., Vol. XV, 1886, p. 581.
‡ "Physical Data and Statistics", William Hammond Hall, M. Am. Soc. C. E., State Engineer of California, 1886.

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At Kingsburg 3-ft. square pans were used, one on land and one floating in Kings River. During the first year the land pan was on a railroad trestle, thereafter on the land with earth banked around it to above the water surface. The pans were about 15 in. deep, and the depth of the water in the pans was about 10 in.

The observer was instructed to replenish the water in the pans by the use of a cup whenever it was found to be about ¼ in. below the top of a pointed peg in the center of the pan. The cup used was dimensioned to represent 0.01 ft. in depth. No attempt was made to establish the daily rate of evaporation.

The results of the Kingsburg studies are summarized in Table 68 and the curve therefrom is shown in Fig. 28. The 4-year record showed 3.85 ft. as the average annual depth of water evaporating from the pan floating in the river, and 4.96 ft. as that from the pan on the ground.

The results of the observations at Lee Bridge and at Boston are as shown in Table 69 and Fig. 29.

For comparison, the results of evaporation observations by Charles H. Lee, Assoc. M. Am. Soc. C. E., in Owens Valley, California, made for the Department of Public Works, Bureau of the Los Angeles Aqueduct, Los Angeles, in co-operation with the U. S. Geological Survey and the State of California, may also be referred to. These results have been published,* and have been used as the basis for Fig. 30, in which they will be found comparable with other studies of the relation betwen temperature and evaporation.

The Owens Valley pan was 3 ft. square and 10 in. deep; it was floating for a time in the water of Blackrock Slough and later in Owens River at the Citrus Bridge, about 4 miles east of Independence.

"The depth of water beneath the pan varied from 1 to 5 ft., depending on the river stage. The river had a moderate velocity and varied in temperature from about 75° Fahr. in summer to about 40° Fahr. in winter."

Mr. Lee says that in an embedded land pan, about 3 miles east of Independence, the water temperature ranged from 95° in summer to 32° in winter, and that a comparison month by month shows the excess of evaporation loss from the land pan to have been about 33 per cent. Stated in the usual way, the rate of evaporation from the floating pan was about 75% of that from the land pan.

It was not the purpose of the Kingsburg observations to establish a law of evaporation, but to ascertain the probable annual evaporation from an open water surface. It is believed that the evaporation measured from the pan floating in Kings River was somewhat less than would have been observed from a pan floating on an open water surface, and less, too, than that from a large open water surface,

* *Water Supply and Irrigation Paper No. 294.*

TABLE 68.—EVAPORATION AT KINGSBURG, CAL., NOVEMBER, 1881, TO OCTOBER, 1885. Mr. Grunsky.

Temperatures are based on U. S. Weather Bureau records at Fresno from 1888 to 1902. The climate at Fresno is practically the same as at Kingsburg, which lies in the same great valley, 20 miles to the southeast.

Month.	Temperature, in degrees Fahrenheit.	EVAPORATION IN 24 HOURS, IN FEET.		
		Floating pan.	Land pan.	Open water (probable).
Jan	45.3	0.0021	0.0026	0.0022
Feb	50.2	0.0036	0.0033	0.0035
Mar	54.4	0.0066	0.0073	0.0068
Apr	60.8	0.0071	0.0095	0.0077
May	67.4	0.0091	0.0178	0.0133
June	74.1	0.0161	0.0259	0.0185
July	82.1	0.0203	0.0292	0.0225
Aug	81.0	0.0233	0.0278	0.0244
Sept	73.8	0.0180	0.0205	0.0186
Oct	64.2	0.0109	0.0093	0.0105
Nov	54.6	0.0059	0.0058	0.0059
Dec	47.0	0.0032	0.0034	0.0032

TABLE 69.—EVAPORATION RECORDS AT LEE BRIDGE, ENGLAND, 1860-1873; AND AT CHESTNUT HILL, BOSTON, FOR 6 YEARS, BETWEEN 1876 AND 1886.

Month.	AT CHESTNUT HILL.		AT LEE BRIDGE.	
	Temperature, in degrees Fahrenheit.	Evaporation per 24 hours, in feet.	Temperature,* in degrees Fahrenheit.	Evaporation per 24 hours, in feet.
Jan	27	0.0026	38.5	0.0021
Feb	28	0.0030	40.8	0.0018
Mar	35	0.0039	42.5	0.0029
Apr	45	0.0066	48.1	0.0058
May	57	0.0103	54.2	0.0074
June	66	0.0148	60.2	0.0087
July	72	0.0167	64.0	0.0093
Aug	70	0.0161	63.3	0.0078
Sept	63	0.0135	58.4	0.0045
Oct	53	0.0093	50.6	0.0028
Nov	42	0.0062	43.9	0.0020
Dec	32	0.0037	39.5	0.0017

* The temperature for Boston is from U. S. Weather Bureau records for the period, 1873 to 1903, and the rate of evaporation is from FitzGerald's smoothed out curve.

for the reason that some protection against sun and wind was afforded to this pan by high river banks, a fringe of low trees, and a near-by bridge, and for the further reason that the temperature of the river was probably a little less than that which would have obtained in an open body of water. The land pan, on the other hand, undoubtedly

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lost more water than would have been lost from the surface of a large open water body.

This reasoning led the writer to adopt values determined by giving the floating pan records three times the weight of those of the land pan as a basis for approximating evaporation rates from large bodies of water when mean monthly temperatures in their vicinity are known. He found it convenient to express the results in a diagram which he believes to be fairly dependable under the climatic conditions of the West and Southwest. The writer has some doubt, however (which he would like to see cleared up), concerning the rates indicated by the Kingsburg records at low temperatures. From very cursory comparisons with other records, it seems possible that the values indicated by the Kingsburg diagram at low temperatures are too low. Although such error, if it exists, will be apparent in the evaporation during the winter, it will not greatly affect the annual result, because the proportional evaporation in the winter is small.

It will be noted that the Kingsburg records indicate that the mean annual rate of evaporation from the pan floating in the river is about 78% of that from an embedded land pan, and that the ratio which the writer has accepted as probable between the 3-ft. square land pan at Kingsburg and an open water surface is about 83%, applicable to annual, but not to individual monthly, records.

The relation between the mean monthly temperature and the mean monthly rate of evaporation deduced from the Kingsburg record is as shown in Table 70.

TABLE 70.—RELATION BETWEEN MEAN MONTHLY TEMPERATURE AND MEAN MONTHLY RATE OF EVAPORATION, BASED ON THE KINGSBURG RECORD.

Mean monthly temperature, in degrees Fahrenheit.	EVAPORATION FROM OPEN WATER SURFACE. MEAN MONTHLY RATE PER 24 HOURS.	
	Feet.	Inches.
35	0.0010	0.012
40	0.0016	0.019
45	0.0025	0.030
50	0.0038	0.046
55	0.0055	0.066
60	0.0080	0.096
65	0.0110	0.132
70	0.0147	0.177
75	0.0187	0.225
80	0.0228	0.274
85	0.0270	0.324
90	0.0312	0.374

Although the fact is recognized that the rate of evaporation from a floating pan is not necessarily the same as that from the body of

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water which surrounds the pan, it is believed that, generally, there will be no great departure, and that lack of perfect agreement must be ascribed primarily to differences in water temperature which are ordinarily not large, but which, nevertheless, may cause some variation in the rate of evaporation from the pan and from the surrounding open water. In regions of much sunshine, it is believed that the pan should show a somewhat higher mean annual rate than the open water. It is probable, too, that the sides of the pan—due to capillary action—have the effect of accelerating evaporation, and that this is a contributory cause to the supposed excess of the pan rate over that of the open water. There is another reason, however, why a pan record of evaporation may not correctly represent the evaporation from a large water surface. This is found in the varying exposure to sun and wind of different portions of the water body. Departures due to this cause may be in either direction.

Although this is true, there is no evidence at hand, as already stated, to show that the same ratios will obtain between the evaporation rates from various sized pans floating in the water as apply to similar pans placed on the ground, or above the water surface, and not under temperature control by a large body of water.

The writer does not propose to examine the original data used by Professor Bigelow in determining the results relating to the effect of the size of the pan on the coefficient in the Bigelow evaporation formula, and does not question their reliability, but he desires to point out that the evaporation, according to the formula in which C_2 is a coefficient, is dependent on the water temperature and temperature changes, and that, therefore, the formula cannot be used to show the relation between the evaporation from an open water surface and that from a floating pan, unless water temperature records are available. The writer believes that any conclusion, such as the authors deduce from the Bigelow results—that the evaporation from the open water surface of Lake Conchos was 62% of the evaporation from a 3-ft. square pan floating thereon—should be accepted with caution. This could only be the case if there was a material difference in temperature conditions between the surface water of the lake and the water in the floating pan, which is quite improbable.

Professor Bigelow had no pans floating on Salton Sea,* and therefore the relation between a floating-pan record and the annual evaporation, of about 6 ft., from Salton Sea, was not recorded by him. In view of the assumption now made by the authors—that the evaporation from a 3-ft. floating pan would be so much greater than that from a large open water surface—this is regrettable.

* Professor Bigelow says: "It was not possible to float pans in the waves of Salton Sea * * *". (*Engineering News*, Vol. 63, June 16th, 1910, p. 695.)

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Conclusion (*a*), as stated on page 1843, is probably correct if restricted to land pans (not embedded). They are based on Professor Bigelow's experiments with pans about 10 in. deep, with sides above the water, freely exposed to air and sun, and not cooled by surrounding water, and, for the present, they should be restricted to such pans.

For Conclusion (*c*) the writer can find no basis. By reference to page 1857 it will be seen that, because the coefficient in an evaporation formula based on water temperature and other factors varies from 0.023,* for open water, to about 0.039,* for a 3-ft. circular pan (not square), the authors have assumed this relation to apply to the annual evaporation rate. This latter relation was not established by Professor Bigelow's work.

Conclusion (*d*) on page 1843—that the effect of a change in the mean monthly temperature is the same at low as at high mean monthly temperatures—should be weighed in comparison with such records as those at Chestnut Hill, Mass., at Lee Bridge, England, in Owens Valley, Cal., and at Kingsburg, Cal., all of which indicate a departure from this conclusion.

Although it may be admitted that, for the same mean monthly temperature, the evaporation is greater at high than at low altitudes, the law of this increase is not to be regarded as proven by the data presented in the paper, and Conclusion (*e*) on page 1843 should not be accepted without further verification.

In Table 17 the authors note floating-pan evaporation for 14 stations, of which 11 are Piche evaporimeter calculations, as explained in the table. Though it may happen that at some points the evaporimeter makes possible a close approximation of evaporation from a floating pan, it remains to be demonstrated that the record of the open air instrument may under all circumstances be substituted for, and called equivalent to, a floating-pan record. The conclusions reached by Professor Bigelow in reference to the effect of elevation above ground on the rate of the evaporation from small pans and the unknown position of the Piche instrument with reference to any possible water surface, should invalidate this substitution. Table 17, in the form in which presented, may be misleading, because the acceptance of an interchangeability of results as there made does not seem warranted. Nevertheless, the records in that table show for each station the close inter-relation between the mean monthly evaporation rate and the mean monthly temperature.

On page 1877 there is a reference to curves showing the relations between temperature and evaporation in "3-ft. square floating-pans at five measured evaporation stations on the Great Plateau." Of these stations, two—Albuquerque and Elephant Butte—had no floating

* From *Monthly Weather Review*, July, 1910, p. 1133, by F. H. Bigelow.

pans, and at Carlsbad the record is a composite of an expanded floating-pan record at Lake Avalon and a land-pan record at Carlsbad. The latter is unreliable, and unquestionably indicates too small an annual evaporation rate. It seems to the writer that these records are of but little value in the discussion of the relation between evaporation from floating pans and temperature. Mr. Grunsky.

The results presented by the authors, in the diagram on Plate LXV, for all stations combined, should not be accepted as conclusive, for the reason, already pointed out, that the records on which the diagram is based are not all from floating pans, and for the further reason that the starting point in the matter of time has not been brought into any definite relation to the seasons. In the preparation of such a diagram, a uniform starting point in relation to the calendar year should be used. When this is done for such dependable evaporation records as those at Lee Bridge, England; at Chestnut Hill, Boston; and in Owens Valley, Cal., as also for the somewhat less dependable record at Kingsburg, Cal., it will be found that the curves will all have a pronounced ogee shape, their steepest inclination falling in the spring and summer months. This is due mainly to the fact, elsewhere noted, that at the lower temperature of the winter months the change in the evaporation rate due to each change of a degree in the mean monthly temperature is less than for the change of a degree in the warmer months of summer.

In Tables 38 and 39 the evaporation from pans on and at Lake Conchos is given. The land-pan records are used to deduce floating-pan records by applying a coefficient of 80% to the monthly records. It is probable that this factor fairly represents the relation between the yearly land-pan and the floating-pan evaporation rates, but it will hardly do to accept this as correct for the individual months.

In the winter, from December to February, inclusive, the floating pan will probably ordinarily show, as it does elsewhere, a rate of evaporation as great as, or greater than, that of the land pan, for the reason that the mean water temperature in cold months in the floating pan will be as high as, or higher than, that of the land pan. The probability is that the estimates which are based on Table 38 are too low, because in the 3 months, January to March, the evaporation from the land pan should be about the same as that from a near-by floating pan. Therefore, too much reliance should not be placed on the diagram, Fig. 15, showing the relation at Lake Conchos between the mean monthly temperature and evaporation from the lake's surface. The line there shown should probably be curved, indicating a smaller effect of temperature variation at low than at high temperatures.

The writer believes that the rate of evaporation in relation to mean monthly temperature increases with elevation in some measure,

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as shown by the authors on Table 17. It is reasonable to expect this, not only because the temperature at which water boils falls with increasing altitude, but, also, because the temperature range, particularly the diurnal range, ordinarily increases with altitude. According to the evaporation law, the mean monthly rate of evaporation will be greatest at that place which shows the greatest temperature fluctuations, because each degree of temperature above the mean produces more evaporation than each degree below the mean.

It is notable that the records for Lee Bridge, Chestnut Hill, and Owens Valley all indicate higher rates of evaporation for the spring months than for the fall months. Temperature is not the only controlling factor, and when mean monthly temperatures are used as the basis of an evaporation estimate, the results should be regarded as dependable for the entire year only, and not for the individual months, except possibly under similar climatic conditions. The Kingsburg record did not show the peculiarity of greater evaporation, for any given mean monthly temperature, in spring than in autumn.

On the assumption that the mean monthly temperatures are about as shown in Table 30, for 1913 and parts of 1912 and 1914, the Kingsburg curve indicates a mean annual evaporation from the water surface of Lake Conchos of 60 in. This is subject to correction for higher elevation. (Lake Conchos is at Elevation 4 300 and Kingsburg is about at Elevation 300.) This correction is perhaps but little in the hot summer months, but may affect the result materially in the cold winter months. It is believed that this correction will not be as great as the curves presented in the paper would indicate, but, nevertheless, it may be an increase of from 20 to 30 per cent. It is the writer's opinion that further observations at Lake Conchos will show that the evaporation there will be between the 60 in. indicated by the Kingsburg curve and the 85 in. which the authors have estimated as the annual evaporation from a 3-ft. square pan floating on the water surface of the lake.

The results at Kingsburg, though not recommended as fully dependable, have been tested by the fall of the water surface at Salton Sea, where the check was satisfactory for the year 1907-1908.

If temperatures at Salton Sea for a normal year are approximated from the records at Indio by applying a probable correction, ascertained from the records at Mecca, nearer the Sea, and at Brawley, south of the Sea, they will be found to be about as shown in Table 71.

Although it is probable that the temperatures as noted in Table 71 are somewhat too high when applied to the air above so large a body of water as the Salton Sea, they have been used to estimate—by the aid of the Kingsburg curve—the probable normal annual evaporation from Salton Sea. The evaporation from Salton Sea thus computed, and indicated in Table 71 for a normal year, is 6.24 ft., or 75 in.

TABLE 71.—EVAPORATION FROM SALTON SEA. ESTIMATED FROM MEAN MONTHLY TEMPERATURES AND THE KINGSBURG EVAPORATION CURVE. Mr. Grunsky.

The temperatures at Indio were corrected by comparison with temperatures at Brawley and Mecca for temperatures at Salton Sea.

Month.	Mean monthly temperature, in degrees Fahrenheit.	EVAPORATION:	
		Foot per day.	Total, in feet.
Jan.	50.0	0.0038	0.118
Feb.	55.0	0.0055	0.154
March	62.5	0.0094	0.291
April	69.5	0.0144	0.432
May	77.0	0.0203	0.629
June	85.5	0.0274	0.822
July	91.5	0.0326	1.010
Aug.	90.0	0.0320	0.993
Sept.	83.5	0.0261	0.783
Oct.	72.5	0.0167	0.518
Nov.	59.5	0.0076	0.228
Dec.	53.5	0.0052	0.161
Total			6.239

This is about 12% in excess of the mean annual water loss from Salton Sea, as estimated by Mr. Robson for a 6-year period. Perhaps here, too, some correction should be applied for difference in elevation, Salton Sea being about 500 ft. lower than Kingsburg.

In conclusion the writer desires to repeat his statement of some years ago to Professor Bigelow, that no formula of the Dalton type, or of the type suggested by Professor Bigelow, will meet the requirements of the engineer when called on to determine from weather conditions alone the quantity of evaporation from a sheet of water not yet in existence.

H. A. WHITNEY,* M. AM. SOC. C. E. (by letter).—The Profession certainly is indebted to the authors of this paper for the thorough manner in which they have covered every detail. As the paper comes from engineers of high standing in the Profession, these studies are very valuable. Mr. Whitney.

The writer was Assistant Superintendent and Hydraulic Engineer of the Department of Water of San Diego, Cal., from 1912 to 1915, and is able to add materially to the evaporation data on the Upper Otay Reservoir in Table 57, by Mr. Post.

From the beginning of February, 1913, until the close of December, 1915, continuous gauge readings of the water surface in the reservoir were taken, and these, with the corresponding rise and fall of the reservoir surface, rainfall on the reservoir, and net evaporation losses from it, are given in full in Table 72.

* San Francisco, Cal.

Mr. Whitney.

TABLE 72.—GAUGE-READINGS OF SURFACE, RAINFALL, ETC., AT UPPER OTAY RESERVOIR.

Date.	Gauge reading. (Distance of water below crest of dam.)		Rise (+) and fall (−) of reservoir surface, between dates of Column 1.	Correction for observed rainfall between dates of Column 1. (From Table 73.)	Algebraic sum of Columns 3 and 4; or probable evaporation from reservoir surface between dates of Column 1.	
(1)	(2)		(3)	(4)	(5)	
1913	Feet.	Inches.	Feet.	Feet.	Feet.	Inches.
Jan. 1....	....	....	.....	.....	....	....
Feb. 1....	8	10	+ 2.17	*— 0.32	....	....
Mar. 1....	6	8	+ 0.42	— 0.06	....	....
Apr. 1....	6	3	+ 2.25	— 0.01	....	....
May 1....	4	0	— 0.42	— 0.00	— 0.42 =	5.0
June 1....	4	5	— 0.46	— 0.02	— 0.48 =	5.8
July 1....	4	10⅛	— 0.71	— 0.01	— 0.72 =	8.7
Aug. 1....	5	7	— 0.67	— 0.00	— 0.67 =	8.0
Sept. 1....	6	3	— 0.50	— 0.00	— 0.50 =	6.0
Oct. 1....	6	9	— 0.42	— 0.00	— 0.42 =	5.0
Nov 1....	7	2	— 0.33	— 0.18	— 0.51 =	6.1
Dec. 1....	7	6	— 0.17	— 0.14	— 0.31 =	3.7
1914						
Jan. 1....	7	8	— 0.08			
" 26....	7	9	+ 0.50	— 0.25	....	....
Feb. 1....	7	3				
			+ 0.50			
" 18....	6	9	— 0.04	— 0.18	....	....
Mar. 1....	6	9½	— 0.42			
Apr. 1....	7	2½	— 0.50	— 0.07	— 0.49 =	5.9
May 1....	7	8¼	— 0.44	— 0.06	— 0.56 =	6.7
" 23....	8	1¾	— 0.15	— 0.01	— 0.60 =	7.2
			(— 0.59)			
June 1....	8	3½	— 0.12			
" 13....	8	5	— 0.21			
" 20....	8	7½	— 0.13	— 0.00	— 0.56 =	6.7
" 28....	8	9	— 0.10			
			(— 0.56)			
July 1....	8	10¼	— 0.23			
" 11....	9	1	— 0.17	— 0.00	— 0.73 =	8.8
" 18....	9	3	— 0.33			
			(— 0.73)			

* If rain had not occurred, the surface would have risen less or fallen more.

TABLE 72—(*Continued*).

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Date.	Gauge reading. (Distance of water below crest of dam.)		Rise (+) and fall (—) of reservoir surface, between dates of Column 1.	Correction for observed rainfall between dates of Column 1. (From Table 73.)	Algebraic sum of Columns 3 and 4; or probable evaporation from reservoir surface between dates of Column 1.	
(1)	(2)		(3)	(4)	(5)	
1914	Feet.	Inches.	Feet.	Feet.	Feet.	Inches.
Aug. 1....	9	7				
			— 0.33			
" 15....	9	11	— 0.06	— 0.00	— 0.67 (*a*)	= 8.0
" 26....	9	11¾	— 0.27			
			(— 0.67)			
Sept. 1....	10	3				
			— 0.58	— 0.00	— 0.58 (*b*)	= 7.0
Oct. 1....	10	10	— 0.25			
" 16....	11	1	†— 11.92	— 0.07	....	....
Nov. 1....	23	0				
			— 0.08	— 0.05	— 0.13	= 1.6
Dec. 1....	23	1				
			— 0.92	— 0.21	— 1.13	= 13.6
1915						
Jan. 1....	24	0			....	....
			+ 7.75	— 0.36		
Feb. 1....	16	3				
			+ 7.50	— 0.33	....	....
Mar. 1....	8	9				
			+ 2.25	— 0.07	....	....
Apr. 1....	6	6				
			— 0.04			
" 15....	6	6¼	+ 0.12	— 0.15	....	....
May 1....	6	5				
			— 0.08			
" 5....	6	6	— 0.08			
" 9....	6	7	— 0.09			
" 14....	6	8	— 0.08	— 0.08	— 0.66	= 7.9
" 18....	6	9	— 0.08			
May 22....	6	10				
			— 0.09			
" 26....	6	11	— 0.08	.....	....	....
			(— 0.58)			
June 1....	7	0				
			— 0.08			
" 6....	7	1	— 0.09			
" 11....	7	2	— 0.08			
" 16....	7	3	— 0.08	— 0.00	— 0.50	= 6.0
" 21....	7	4	— 0.09			
" 26....	7	5	— 0.08			
			(— 0.50)			

† Withdrawal for water supply.

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TABLE 72—(*Continued*).

Date.	Gauge reading. (Distance of water below crest of dam.)		Rise (+) and fall (−) of reservoir surface, between dates of Column 1.	Correction for observed rainfall between dates of Column 1. (From Table 73.)	Algebraic sum of Columns 3 and 4; or probable evaporation from reservoir surface between dates of Column 1.	
(1)	(2)		(3)	(4)	(5)	
1915	Feet.	Inches.	Feet.	Feet.	Feet.	Inches.
July 1....	7	6				
" 5....	7	7	−0.08			
" 9....	7	8	−0.09			
" 13....	7	9	−0.08			
" 17....	7	10	−0.08			
" 20....	7	11	−0.09	−0.00	−0.67	= 8.0
" 24....	8	0	−0.08			
" 28...	8	1	−0.08			
			−0.09			
			(−0.67)			
Aug. 1....	8	2	−0.08			
" 5....	8	3	−0.08			
" 9....	8	4	−0.09			
" 13....	8	5	−0.08			
" 17....	8	6	−0.09	−0.01	−0.83	= 10.0
" 21....	8	7	−0.08			
" 25....	8	8	−0.09			
" 29....	8	9	−0.25			
			(−0.84)			
Sept. 1....	9	0	−0.42	−0.00	−0.42	= 5.0
Oct. 1....	9	5	−0.33	−0.00	−0.33	= 4.0
Nov. 1....	9	9	−0.33	−0.06	−0.39	= 4.7
Dec. 1†...	10	1	+0.04	−0.22	....	....
1916						
Jan. 5....	10	0½	....	....	....	....

† Withdrawal for water supply.

Similar gauge readings and rainfalls were taken at Chollas Heights Reservoir, for those months of 1914 and 1915 during which it is believed there was no inflow or outflow at that reservoir. The reservoir at Chollas Heights is used for reserve and for regulating the distributing system of San Diego. It is about 4 miles east of the eastern boundary of San Diego, and is fed from the 30-in. pipe leading from the Lower Otay Reservoir, which pipe conducts the run-off waters from the various drainage areas of the San Diego water system.

TABLE 73.—RAINFALLS NEAR UPPER OTAY AND CHOLLAS HEIGHTS RESERVOIRS. Mr. Whitney.

Date.	NEAR UPPER OTAY RESERVOIR.		NEAR CHOLLAS HEIGHTS RESERVOIR.	
	Depth, in inches.	Depth, in feet.	Depth, in inches.	Depth, in feet.
1913				
Jan.	1.51	0.13		
Feb.	3.89	0.32		
Mar.	0.67	0.06		
Apr.	0.15	0.01		
May	0.04	0.00		
June	0.23	0.02	No rain gauge readings taken.	
July	0.10	0.01		
Aug	0.05	0.00		
Sept	0.00	0.00		
Oct.	0.01	0.00		
Nov.	2.17	0.18		
Dec.	1.63	0.14		
	(10.45)	(0.87)		
1914				
Jan.	2.98	0.25	3.48	0.29
Feb.	2.17	0.18	2.51	0.21
Mar	0.85	0.07	0.61	0.05
Apr	0.78	0.06	1.05	0.09
May	0.15	0.01	0.04	0.00
June	0.00	0.00	0.16	0.01
July	0.00	0.00	0.00	0.00
Aug	0.00	0.00	0.00	0.00
Sept	0.00	0.00	0.00	0.00
Oct.	0.08	0.07	1.47	0.12
Nov	0.63	0.05	1.11	0.09
Dec.	2.58	0.21	3.22	0.27
	(10.94)	(0.90)	(13.69)	(1.13)
1915				
Jan.	4.28	0.36	5.34	0.45
Feb.	3.97	0.33	4.71	0.39
Mar	0.87	0.07	0.74	0.06
Apr	1.79	0.15	1.74	0.15
May	1.00	0.08	0.86	0.07
June	0.00	0.00	0.01	0.00
July	0.00	0.00	0.00	0.00
Aug	0.15	0.01	0.00	0.00
Sept	0.00	0.00	0.00	0.00
Oct.	0.00	0.00	0.00	0.00
Nov	0.76	0.06	1.31	0.11
Dec.	2.68	0.22	1.19	0.10
	(15.50)	(1.28)	(16.00)	(1.33)

The Chollas Heights Reservoir is frequently drawn upon for reserve water, and a preliminary diagram of its gauge heights showed so many fluctuations of its surface in 1915, from accessions and uses, that only the 1914 records (when neither accessions nor uses were believed to have occurred) apparently were applicable to this discussion. Again, however, a tabulation of the 1914 Chollas Heights records similar to those in Table 72 showed monthly evaporation depths which, almost without exception, were so large as to be unreasonable or absurd; hence, the Chollas Heights gauge readings are dismissed from further consideration, as being unreliable data for evaporation. Merely as a surmise, it is supposed that at times water was withdrawn from that

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reservoir without the order or knowledge of the writer, or that the regulating valve was not entirely closed.

The rainfalls of Table 72 were measured in a standard 8-in. Weather Bureau rain gauge, near the Lower Otay Dam, about 3 miles down stream from the Upper Otay Reservoir. There was a similar rain gauge near the Chollas Heights Reservoir, nearly 12 miles from the Lower Otay rain gauge, and, for comparison, the monthly rainfalls by the two gauges are given in Table 73.

From the close agreement between both the monthly and seasonal rainfalls at the Lower Otay and Chollas Heights rain gauges (and from the rainfalls at Sweetwater Reservoir given in Table 57), it is evident that no material errors can be introduced by using the Lower Otay rainfalls in connection with the Upper Otay Reservoir, especially for the practically rainless months for which the evaporation depths are given in Table 72. The monthly evaporation depths of Table 72 are summarized and compared in Table 74.

The Upper Otay Reservoir is about 13 miles southeast of San Diego and about 550 ft. above the sea. It is approximately 1 mile long and ½ mile wide. At gauge height, 3 ft. below top of dam, it has an area of 160 acres and an average depth of 20.7 ft.; 10 ft. lower its area is 115 acres and its average depth 16.8 ft. It is essentially a storage reservoir, and both accessions to it and withdrawals from it are comparatively infrequent.

As a general rule (with recent very notable exceptions) the rainfalls near San Diego are infrequent and small, those of many of the months causing no run-offs or stream flows; and, during the months included in Table 74, the writer believes, from his knowledge of the local conditions, that there were no material accessions to the waters in the Upper Otay Reservoir except the small rainfalls on its surface, no material withdrawals for water-supply uses, no losses from visible seepage, and probably no material losses from absorption by the bottom of the reservoir and additions to the ground-water. There are about 3 acres of tules or cattails in the reservoir, which, by their transpiration, probably increase the evaporation losses slightly; but, because of their small proportionate area, their influence on the evaporation depth for the whole reservoir cannot be material.

Hence, it is believed that the evaporations of Table 74 may be accepted as fairly reliable values for the evaporation depths from the Upper Otay Reservoir for the months and years given.

From Table 56, showing the monthly percentages near San Diego of the total yearly evaporation, the total evaporation depths of Table 74 should be the following percentages of the total yearly depth:

1913—May-December, incl.....79%
1914—March-October, incl.....82%
1915—May-December, incl.....75%

TABLE 74.—MONTHLY EVAPORATION DEPTHS, UPPER OTAY RESERVOIR, IN INCHES. Mr. Whitney.

Month.	1913.	1914.	1915.	Mean.	Range.
March	....	5.9	....	(1) 5.9	.....
April	....	6.7	....	(1) 6.7	.....
May	5.0	7.2	7.9	(3) 6.7	(5.0 to 7.9)
June	5.8	6.7	6.0	(3) 6.2	(5.8 to 6.7)
July	8.7	8.8	8.0	(3) 8.5	(8.0 to 8.8)
August	8.0	8.0	8.0	(3) 8.0	(8.0 to 8.4)
September	6.0	7.0	5.0	(3) 6.0	(5.0 to 7.0)
October	5.0	5.0	4.0	(3) 4.7	(4.0 to 5.0)
November	6.1	....	4.7	(2) 5.4	(4.7 to 6.1)
December	3.7	....	....	(1) 3.7	(.....)
	48.3	55.3	43.6	(10) 61.8	

Hence, from Table 74 and the percentages as taken from Table 56, the total yearly evaporation depths should be about as in Table 75.

The total yearly evaporations for the 3 years vary rather widely from each other, having an extreme variation of 9.3 in. Assuming the mean for the 3 years as 62.2 in., the evaporation depth for 1913 was 2% below the mean, that for 1914 was 8% above it, and that for 1915 was 6.7% below it.

TABLE 75.—PROBABLE NET EVAPORATION, IN INCHES, FOR 1913, 1914, AND 1915, FROM THE WATER SURFACE OF UPPER OTAY RESERVOIR.

Month.	1913.	1914.	1915.	Mean.	Percentage.
January	2.5*	2.7*	2.4*	2.5 +	4.0
February	3.0*	3.4*	2.9*	3.1	5.0
March	3.0*	5.9	2.9*	3.9 +	6.3
April	4.3*	6.7	4.1*	5.0 +	8.0
May	5.0	7.2	7.9	6.7	10.7
June	5.8	6.7	6.0	6.2 +	10.0
July	8.7	8.8	8.0	8.5	13.6
August	8.0	8.0	8.0	8.0	12.9
September	6.0	7.0	5.0	6.0	9.6
October	5.0	5.0*	4.0	4.7 +	7.6
November	6.1	3.4*	4.7	4.7 −	7.6
December	3.7	2.7*	2.3*	2.9	4.7
	61.1	67.5	58.2	62.2	100.0

* The evaporations for these months were calculated from the percentages shown in Table 56. The evaporation for October, 1914, is estimated from the 16th to the 31st.

From Table 56, the yearly evaporation depths from the Sweetwater Reservoir (5 miles from the Upper Otay Reservoir) for the 5 years, 1910 to 1914, varied from 45.76 to 63.32 in., with a mean of 56.98 in. So far as can be judged by the 3 years' observations there, the mean yearly evaporation depth from the Upper Otay Reservoir should be approximately 62.2 in. However, evaporations were measured from the water surfaces for 1913 and 1914 at both the Upper Otay and Sweetwater Reservoirs, with the following comparative results:

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The evaporation from the Sweetwater Reservoir from April 30th to December 31st, 1913, was 45.23 in., and at the Upper Otay from April 30th to December 31st, 1913, it was 48.3 in., making the evaporation from the Sweetwater Reservoir 93½% of that from the Upper Otay.

From March 1st until October 31st, 1914, the evaporation from the surface of the Sweetwater Reservoir was 41.02 in., and that from the water surface of the Upper Otay Reservoir for the same period (from May 1st to October 31st, the last half month being approximated) was 55.3 in., thus making the evaporation from the water surface of the Sweetwater Reservoir 74.1% of that from the Upper Otay Reservoir.

As the two reservoirs are only 5 miles apart, and their evaporation conditions apparently do not differ materially, the wide variation in their evaporation depths for 1914 throws some doubt on the accuracy and reliability of the evaporation records of that year at one or both of the reservoirs. No account was taken of the gauge heights subsequent to letting out the water from the Upper Otay Reservoir on October 17th, 1914. As already mentioned, the evaporation of the last 2½ months of that year was estimated from the percentages given in Table 56.

On October 16th, 1915, daily readings were begun with a 3-ft. square evaporation pan floating on the Lower Otay Reservoir, and these measurements were continued until the Lower Otay Dam failed, on January 27th, 1916.

As shown by the notes, the replenishing water was measured daily only to the nearest quart, a quart equalling 0.04448 in. depth in the 3-ft. square pan. Of course, the even quart or quarts added each morning sometimes fell a little short of the top of the sharpened pin in the pan, and sometimes overran it slightly, and thus introduced more or less error into the daily evaporations; but it is evident that for a number of quarts, as for a month, the error thus introduced is negligible. The pan evaporations thus measured were as follows:

For all October, 1915 = 5.0 in. (2½ in. measured Oct. 16-31).
" " November, " = 3.3 " (8.3 in. cumulative).
" " December, " = 4.4 " (12.7 in. cumulative).

From Table 75, the evaporations from the reservoir during the same months were as follows:

For all October, = 4.0 in.
" " November, = 4.7 " (8.7 in. cumulative).
" " December, = 2.3 " (11.0 in. cumulative).

The comparisons of the evaporations from the reservoir with those from the pan are given in Table 76.

In reference to some of the other reservoirs mentioned by Mr. Post, the writer has a slight personal knowledge of the conditions at the

TABLE 76.—EVAPORATIONS FROM THE UPPER OTAY RESERVOIR, AS PERCENTAGES OF THE EVAPORATIONS FROM A 3-FT. SQUARE PAN FLOATING THEREON. Mr. Whitney.

Monthly.				Cumulative.			
Month.	Reservoir.	Pan.	Percentage.	Period.	Reservoir.	Pan.	Percentage.
	Inches.	Inches.			Inches.	Inches.	
Oct.....	4.0	5.0	80	Oct.	4.0	5.0	80
Nov.....	4.7	3.3	142	Oct. and Nov.	8.7	8.3	105
Dec.....	2.3	4.4	52	Oct., Nov., and Dec.	11.0	12.7	87

Murray Hill Reservoir, and believes there is quite a little seepage from it, which (if corrected for) would make the evaporation from it somewhat less than the 1.60 in. shown on page 1914.

In reference to the comparisons between the evaporations from the Cuyamaca Reservoir and from a floating pan in Table 58, it is believed that (because of the flatness of the land around the reservoir) comparatively large run-offs may be fed into it during rainy months by underwater springs and rock crevices, which accessions no run-off records can show.

M. HEGLY,* ESQ. (by letter).—The writer has examined this paper with much interest and profit, and believes it to be a most valuable contribution to the study of the subject of evaporation from the surfaces of lakes and reservoirs. The results are in remarkable accord with those obtained by other observers, especially with those of Professor Bigelow at Salton Sea. The great care exercised in obtaining the data makes them most valuable. Mr. Hegly.

It is a well-known fact that, until quite recently, the figures given in special treatises and engineering handbooks, on the subject of the quantity of water evaporated from small basins, have not been applicable to large bodies of water, such as lakes, storage ponds built for irrigation works, hydraulic power, or for supplying water to navigation canals. That the figures were too large was well known, but there have been no accurate data which would enable one to calculate with sufficient approximation the actual depth of evaporation which would reduce, in reasonable proportion, the volume of water impounded. There was lacking a rule to be applied to any particular case for using the data obtained from observations, necessarily made on very small surfaces, either by using the formulas given in works on physics or the results obtained in other localities.

Having occasion to solve a similar problem in Tunis during the construction of a large reservoir near the city of that name, the writer

* Ingénieur en Chef, Ponts et Chaussées, Departement de la Haute-Marne, Chaumont (Haute-Marne), France.

Mr. Hegly. assumed that the depth of the annual evaporation would reach 1.60 m. (63 in.); this figure was based on guesswork, however, as the writer does not pretend that he has the perspicacity of Mr. Freeman, who, in 1911, before any local observations and studies had been made, fixed the evaporation from Lake Conchos at 1.40 m. (55 in.).

The only data at the writer's disposal were those given by Professor Ginestous, who, in his "Etudes sur le Climat de la Tunisie", fixes the annual depths of evaporation at 2.255 m. (88.8 in.), a value evidently much too large; for June, July, and August he determined the value 0.72 m. (28.3 in.), a figure which the writer has verified by observations on vats, 4 m. (13.12 ft.) in diameter, placed on the site of the proposed reservoir. It may be of interest to note that during the sirocco a daily evaporation of from 13 to 16 mm. ($\frac{1}{2}$ to $\frac{5}{8}$ in.) was observed.

The writer also had at his disposal data relative to the depth of evaporation from the surfaces of large stretches of water near the shores of the Mediterranean observed by M. Salles, *Inspecteur Général, Ponts et Chaussées,** who, as early as 1883, was led to believe that the depth of the annual evaporation at Arles, at the northerly point of the plains of Carmague, 25 km. (15½ miles) from the sea-coast, was 1.05 m. (41.3 in.), and prior observations made by the meteorologists fixed the annual evaporation at 1.876 m. (73.8 in.) for Orange and 2.563 m. (100.9 in.) for Arles.

Very careful studies by the engineers of the *Ponts et Chaussées* in Algeria also supplied valuable data; and their long series of experiments at the storage reservoirs of Djebel-Ouach, which supply the City of Constantine (at an elevation of 650 m. (2 132 ft.) and 65 km. (40 miles) from the sea) determined the depth of annual evaporation for the vicinity of that city to be 0.85 m. (33.5 in.) during the winter and 1.40 m. (55 in.) during the summer, or equal to an annual depth of 2.25 m. (88.5 in.). Undoubtedly, this quantity is too large, and must be corrected for seepage, which cannot be neglected at this point. M. Thévenet, *Directeur de l'École des Sciences d'Alger,* in "l'Essai de Climatologie Algérienne", has reduced this figure to 2 m. (78¾ in.). The writer's observations, however, lead him to believe that 1.60 m. (63 in.) is not far from the correct figure, if the altitude, the mean temperature, and the distance from the sea-coast of the country in the vicinity of Tunis are taken into consideration.

M. Thévenet, after some very elaborate experiments to determine the evaporation in the shade as well as in the sun, reached the following conclusions:

In the shade (direct measurement).....1.014 m. (39.9 in.)
Observed with the Piche evaporimeter.....1.533 m. (60.3 in.)
In the sun (direct measurement).....1.917 m. (75.5 in.)

* *Annales des Ponts et Chaussées*, 1911, Vol. II, p. 425.

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If one is satisfied to use the results of the Piche evaporimeter, he can calculate the evaporation from the surface of a pan similar to those used by M. Thévenet by multiplying by the ratio $\frac{1.917}{1.533}$. From this there is then to be computed the evaporation from the surface of a large reservoir, and the conclusions reached by Messrs. Duryea and Haehl will be of great value for such calculations.

The writer considers as most useful the results obtained from the evaporating pans, both floating and placed on the ground, and especially valuable the percentage, 62 (Conclusion (*c*)), which must be applied to the depth of evaporation from a floating pan in order to get the evaporation from the surface of a large reservoir. On first thought, one is surprised that such a large reduction as 62% must be made, as one would naturally think that the depth of evaporation from the surface of the lake would be the same as that in the pans. There can be no doubt, however, as to the accuracy of this quantity, as the paper has considered the problem from all points of view, and, as the work has been thoroughly checked, the results can be used with confidence, provided the apparatus are of the same shape and dimensions as those described and are set in the same way.

Conclusions (*d*) and (*e*) are most useful, as they show how the evaporation varies with the temperature and the altitude, enabling one to determine the probable evaporation at any place by using data obtained by direct observation in other regions. Diagrams (*a*) and (*b*), Fig. 4, will be of great use for such calculations. They can also be used with Professor Bigelow's formula, by writing $C_2 = 0.024$, which seems to be quite exact.

It is of great value to know how the annual depth of evaporation is divided throughout the months, as such information will enable the observer to limit his observations to a few months of the year, and, in regions similar to northern Africa, in particular, where it seldom rains during the summer, it is most important to know the depth of evaporation during the dry months, which is not compensated for by the rainfall, the precipitation during the winter being about equal to the evaporation. A comparison of the percentages observed at Lake Conchos (page 1849) and those deduced from the figures by M. Salles, obtained at Arles, is made in Table 77.

The percentages of the depth of evaporation during the summer are higher at Arles than at Lake Conchos, and this should be the case in all regions with hot and dry summers.

As a further proof of the statement in the paper in regard to the retarding influence of the white vapor on the surface of the water, one can refer to a phenomenon observable in winter, which is the

Mr. Hegly. reverse of evaporation, namely the surface condensation following a cold spell, which lowers the temperature of the water to 0° cent.; then, if this cold snap is followed by a rise in temperature with an increase in the humidity of the air, the aqueous vapor contained in the atmosphere will condense at the surface of the water, and without any precipitation a slight rise in the water level can be observed. M. Angot, *Directeur du Bureau Central Météorologique de France,* was the first to note this phenomenon.*

TABLE 77.

Month.	Lake Conchos.		Arles.	
	Height, in centimeters.	Percentage.	Height, in centimeters.	Percentages.
January	6.6	4.8	3	2.9
February	7.9	5.6	4	3.8
March	10.4	7.4	7	6.7
April	13.2	9.4	8	7.6
May	16.0	11.5	12	11.4
June	16.3	11.7	14	13.3
July	17.0	12.2	18	17.1
August	15.5	11.1	15	14.3
September	13.2	9.4	10	9.5
October	10.7	7.7	7	6.7
November	7.1	5.1	4	3.8
December	5.8	4.1	3	2.9
Totals	1.397 m., or 55 in.	100	1.05 m., or 41 in.	100

The conclusion reached by Professor Bigelow, and quoted by Mr. Robson, that the evaporation from a slightly brackish water, as that of Salton Sea, is about 2% less than that from fresh water, is correct. As early as 1850, it was noticed that near Marseilles the evaporation from salt-water pans was from 4 to 6% less than that from fresh-water pans.

Mr. Horton. Robert E. Horton,† M. Am. Soc. C. E. (by letter).—On reading this paper the writer was disappointed to find that no records of vapor pressure were kept during the somewhat brief experiments on which the authors base their rather elaborate conclusions, in part at least. The authors present numerous diagrams intended to illustrate the variation of evaporation coincident with varying temperature. They infer that although there are numerous factors affecting evaporation and with which it is more or less correlated, the predominant correlation is with temperature. It has long been known that measured evaporation losses from water surfaces could be represented graphically in a fairly satisfactory manner in terms of temperature alone as the

* *Annales du Bureau Central, Mémoires,* 1894, p. 120.
† Albany, N. Y.

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independent variable. Diagrams similar to those appearing in the paper have been presented, for example: by C. E. Grunsky, M. Am. Soc. C. E.,* by Philip Á Morley Parker,† M. Am. Soc. C. E., and by Adolph E. Meyer,‡ M. Am. Soc. C. E., who gives several such diagrams.

For the purpose of determining the relation of evaporation to temperature, considering the latter as the only independent variable, the writer has tabulated the experimental data for Albuquerque, N. Mex., from the records of the Hadley Climatological Laboratory, as appearing on Plate LXII, and has computed therefrom the correlation coefficient, using the formula:

$$K_C = \frac{\Sigma\,(\Delta_T\ \Delta_E)}{\sqrt{\Sigma\,(\Delta^2_T)\ \Sigma\,(\Delta^2_E)}},$$

where Δ_T = the difference between each observed temperature and the mean temperature,

and Δ_E = the difference between each observed evaporation and the mean evaporation.

The computations are given in Table 78, and indicate a correlation coefficient, 0.92 in this instance. The probable error of a single observation by the usual formula is 0.0189. If there is perfect correlation between two quantities, the correlation coefficient should be unity. If there is no correlation, it would be zero. If the correlation coefficient is five to six times as great as the probable error and is also greater than one-half or two-thirds in absolute value, it indicates a considerable degree of correlation. This calculation appears to indicate that evaporation in the long run may be treated as though it were a function of temperature alone at a given locality. It is true, however, that in any individual period, as for a given month or year, the evaporation is very profoundly modified by other factors, such as variation in the wind velocity and vapor pressure in the air.

The writer does not intend to offer any discussion of that portion of the paper which relates to the fundamental object which the authors had in mind, namely, to determine the mean annual evaporation loss from Lake Conchos, nor does he desire to be taken as concurring or acquiescing in either the methods or conclusions in that portion of the paper. Entirely aside from that, the work done in attempting a correlation of evaporation with temperature and altitude represents a start in a much needed investigation, and contains suggestions which may be useful in solving the vexed problem of the relation of evaporation to altitude.

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* *Engineering News*, August 13th, 1908.

† "The Control of Water", 1913, p. 199.

‡ "Computing Run-off from Rainfall and Other Physical Data", *Transactions*, Am. Soc. C. E., Vol. LXXIX, p. 1056.

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TABLE 78.—CORRELATION OF EVAPORATION AND TEMPERATURE, ALBUQUERQUE, N. MEX.

(1) T	(2) Δ_T	(3) Δ^2_T	(4) E	(5) Δ_E	(6) Δ^2_E	(7) $(\Delta_T\ \Delta_E)$	(8) Month.
40.6	+ 14.9	222.01	2.63	+ 4.56	20.79	+ 67.94	Feb., 1900
49.1	+ 6.4	40.96	6.17	+ 1.02	1.04	+ 6.53	Mar
52.0	+ 3.5	12.25	6.82	+ 0.37	0.14	+ 1.30	Apr.
67.0	− 11.5	132.25	10.08	− 2.89	8.35	+ 33.28	May
76.3	− 20.8	432.64	12.63	− 5.44	29.59	+ 113.15	June
79.1	− 23.6	556.96	11.78	− 4.59	21.07	+ 108.32	July
75.0	− 19.5	380.25	10.21	− 3.02	9.12	+ 58.89	Aug.
67.7	− 12.2	148.84	8.00	− 0.81	0.66	+ 9.88	Sept.
56.8	− 1.3	1.69	4.38	+ 2.81	7.90	− 3.65	Oct.
44.7	+ 10.8	116.64	1.73	+ 5.46	29.81	+ 58.97	Nov.
34.6	+ 20.9	436.81	1.40	+ 5.70	32.49	+ 119.13	Dec., 1900
35.4	+ 20.1	404.01	2.04	+ 5.15	26.52	+ 103.52	Jan., 1901
36.3	+ 19.2	368.64	1.81	+ 5.38	28.94	+ 103.30	Jan., 1903
34.6	+ 20.9	436.81	2.07	+ 5.12	26.21	+ 107.01	Feb.
47.2	+ 8.3	68.89	5.21	+ 1.98	3.92	+ 13.43	Mar.
56.5	− 1.0	1.00	10.05	− 2.86	8.18	+ 2.86	Apr.
62.4	− 6.9	47.61	10.98	− 3.79	14.36	+ 26.15	May
68.4	− 12.9	166.41	11.33	− 4.14	17.14	+ 53.41	June
75.2	− 19.7	388.09	12.36	− 5.17	26.73	+ 101.85	July
74.2	− 18.7	349.69	11.73	− 4.54	20.61	+ 84.90	Aug.
64.0	− 8.5	72.25	9.65	− 2.46	6.05	+ 20.91	Sept.
53.9	+ 1.6	2.56	6.62	+ 0.57	0.32	+ 0.91	Oct.
42.4	+ 13.1	171.61	4.21	+ 2.98	8.88	+ 39.04	Nov.
33.7	+ 21.8	475.24	1.88	+ 5.31	28.20	+ 115.76	Dec., 1903
36.4	+ 19.1	364.81	2.28	+ 4.91	24.11	+ 93.78	Jan., 1904
50.8	+ 4.7	22.09	3.92	+ 3.27	10.69	+ 15.37	Feb.
57.2	− 1.7	2.89	7.48	− 0.29	0.08	+ 0.49	Mar.
58.2	− 2.7	7.29	10.71	− 3.52	12.39	+ 9.50	Apr.
65.0	− 9.5	90.25	12.53	− 5.34	28.51	+ 50.73	May
71.3	− 15.8	249.64	13.03	− 5.84	34.10	+ 92.27	June, 1904
M. 55.5	371.6 Σ	6 171.08	*M.* 7.19	109.29 Σ	486.9 Σ	+ 1 598.9	

The authors are clearly in error in their statement on page 1835 that "Evaporation data in Mexico were entirely lacking." A record of evaporation from two evaporimeters, one in the open, the other in the shade, has been maintained for a number of years by Mr. Manuel E. Pastrana, Director of the Central Meteorological Observatory, the City of Mexico. As this station has an altitude of 7 450 ft. above tide, the results would appear to have been of considerable value in the preparation of this paper, especially in the matter of the relation of evaporation to altitude.

The meteorological data mentioned comprise records by months, for 1895 to 1901, inclusive, for the following: Reduced mean barometer, mean air temperature in the shade and in the open, mean temperature of the soil 2.79 ft. deep, and of water, mean vapor tension in the shade and in the open, mean daily evaporation in the shade and in the open, mean wind velocity and precipitation; and, for 1902 to 1904, inclusive, the mean air temperature, relative humidity, precipitation, and evaporation in the shade and in the open.*

* Tables containing these data have been filed in the Library of the Society, and copies may be obtained by any one interested.

Mr. Horton.

Unfortunately, the writer has not had opportunity to compile these records for the years subsequent to 1904. It will be noted that they include, not only the evaporation from water surfaces under two different conditions, but all the attendant meteorologic data necessary for an interpretation of the results. The evaporation pans are each 8.86 in. in diameter and 4 in. deep. The one in the open stands on the flat roof of a house, the one in the shade is in a corridor with window blinds. The anemometer is on the roof of the house, at a height a little greater than that of the exposed evaporimeter. The pans are filled each morning to a definite level. The water temperature is taken in a large tank or water reservoir, and not in the evaporation pan.

In addition to these extensive and carefully maintained records of evaporation in Mexico, a number of other evaporation stations were established, and similar records of evaporation kept. The writer has at hand the detailed results at the following stations for 1904: Guanajuato (Gto.), Toluca (Mex.), Morelia (Mich.) (Seminario), San Juan Bautista (Tab.), Puebla (Pue.) (Colegio del Estado), Queretaro (Qro.) (Colegio Civil), Merida (Yuc.), Guadalajara (Jal.), Pachuca (Hgo.), Mazatlan (Sin.), Chignahuapam (Pue.), Leon (Gto.), Chihuahua (Chih.), Zacatecas (Zac.), Vera Cruz (Ver.) (S. Juan de Ulua).

The authors quote Professor Bigelow's formula in metric units and give certain coefficients in connection therewith, but, apparently without making any use of this formula in the paper. Those who wish to examine the results of Professor Bigelow's investigations on evaporation should consult *Boletin No. 2,* Oficina Meteorologica Argentina, 1912, or Bigelow's recently issued "Treatise on Meteorology, etc." In these later works, many of Bigelow's tentative conclusions appearing in the earlier papers cited by the authors and others in this paper have been rejected or modified.

The work of the authors may be classed as almost wholly empirical. Far-reaching extrapolations are made from somewhat limited data. Methods are often justified in investigations for specific purposes where it is absolutely necessary to reach some conclusion, but are not justifiable in any attempt to arrive at broad scientific generalizations. The writer feels that the subject of evaporation may be attacked to better advantage by one first having grounded himself thoroughly in the physical principles underlying the phenomena, and thereafter following these principles as closely as possible and doing away with wholly empirical methods where practicable.

In this paper the specific result seems first to have been arrived at by guesswork. The authors, by repeatedly calling attention to this guess, as if it in reality was of some value as corroborating their results, have, unfortunately, and no doubt unintentionally, created

Mr. Horton. the appearance that their investigation was for the purpose of corroborating the original guess. It is a fact that any physical quantity capable of direct measurement, even though its value in a particular instance is wholly unknown, will very often lie close to the mean of three or four good guesses made independently by men of some experience in measuring the quantity in question. It does not follow that guesswork should be substituted for measurement, and, as a general rule, where scientific results are to be obtained, it seems preferable to avoid if possible pre-influencing the mind by having any fixed figure set up in advance.

Mr. Ledoux. J. W. LEDOUX,* M. AM. SOC. C. E. (by letter).—The writer has looked over this paper, but has not had time to study it carefully or even read all the other discussions which, no doubt, are very valuable, but is impressed with the idea that the conclusions in the paper may be seriously misleading.

In 1888 and 1889 the writer made weather observations, which included the measurement of evaporation on a water surface containing about 25 acres, in the Allegheny Mountains. Observations were made at least once a day. The apparatus consisted of a 6-in. pipe, 2 ft. long, with a wooden collar at the top forming a float. A shallow wooden box, some 4 or 5 ft. square, with an 8-in. hole in the center, was provided, and this formed an outside shield to hold the 6-in. pipe vessel. The whole apparatus was floated in the water and anchored at various points in the reservoir. The object of the box was to prevent, as much as possible, the wind from emptying the evaporating vessel. With the aid of a boat, measurements were then made with a hook-gauge from a fixed point at the top of the collar. The evaporating vessel was kept as full as possible, say, within $\frac{1}{2}$ in. of the top. Occasionally, the results were lost on account of wind and rain.

In order to find out whether the results obtained in the reservoir had any relation to those on land, evaporating vessels, each consisting of a length of 18-in. terra cotta pipe with water-tight bottom, were sunk in the ground, so that their top surfaces were level with its surface and slightly above the level of the water in the reservoir, and having about the same exposure as the reservoir itself. Observations on these were continued for some months, simultaneously with barometer, wet-and-dry-bulb thermometer, direction and intensity of wind, sunlight, etc.

Up to that time, the writer was under the impression that the evaporation from water surfaces was more directly related to relative humidity than to any other factor, but one month's observations absolutely dispelled this belief.

* Philadelphia, Pa.

Mr. Ledoux.

The final conclusions were that evaporation depends almost entirely on the temperature and the wind, and has very little relation to humidity, even where the relative humidity varies between 70 and 95%; that land gauges give the same results as floating gauges, provided the temperature of the water in the evaporating vessel is kept the same as that of the water in the reservoir; that a gauge 6 in. in diameter will show as much evaporation as one 18 in. in diameter;

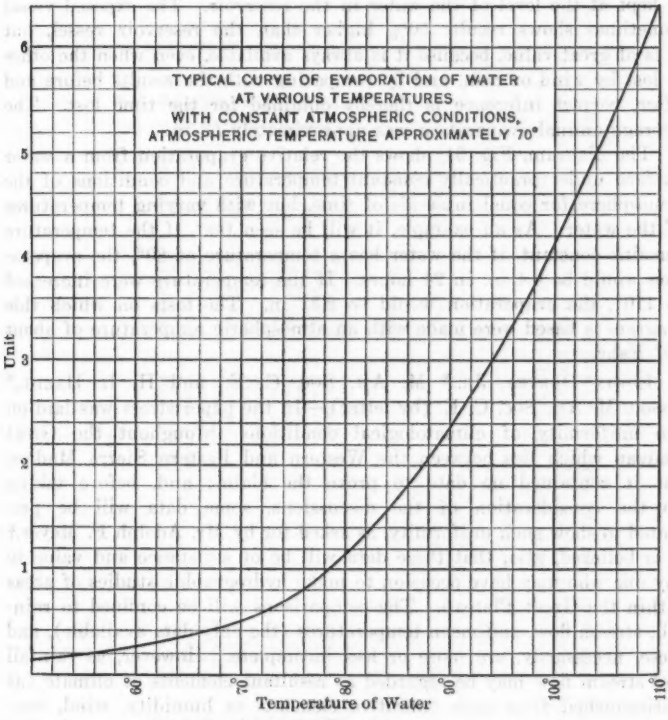

FIG. 31.

that it is important that the evaporating vessel be kept full, so as to get a complete water exposure; that the best way to measure the evaporation is to fill the vessel with a pipette at the same time every day, so that the surface of the water will come up to the same point or hook. The number of inches of evaporation can then be determined by a constant multiplied by the number of cubic centimeters poured into the evaporating vessel.

Mr. Ledoux.

The writer is under the impression that the evaporation from a water surface is affected materially by humidity where the relative humidity is less than 70 per cent.

In one series, near Charleston, S. C., kept under the direction of the writer from 1903 up to the present time, there are two evaporating gauges. One is exposed to the air, about 5 ft. above the ground and covered with a glass 1 or 2 in. above the top of the vessel; the other is kept at the level of the water in the reservoir. The exposed vessel sometimes shows results 50% higher than the reservoir vessel, but it is of great value, because it is always available, even when the other is lost by wind or rain, and by comparing with the results before and after, correct inference is thereby obtained for the time lost. The average annual evaporation was approximately 53 in.

The diagram, Fig. 31, shows the relative evaporation from a water surface under practically constant temperature and conditions of the atmosphere for equal intervals of time, but with varying temperatures of the water. As an example, it will be seen that, if the temperature remains constant, if the water has a temperature of 80°, the evaporation would be 0.4 in. in 24 hours. If the temperature were increased to 110°, the evaporation would be 2.31 in. The tests on which this diagram is based were made with an atmospheric temperature of about 70° Fahr.

Messrs. Duryea and Haehl.

EDWIN DURYEA, JR.,* M. AM. SOC. C. E., and H. L. HAEHL,* ASSOC. M. AM. SOC. C. E. (by letter).—In the paper stress was laid on the uniformity of climatological conditions throughout the Great Plateau which lies between the Western and Eastern Sierra Madres, but it contained no data to prove the claim; and, before taking up the consideration of the discussions, some data will be presented to show such uniformity, as asked for by Mr. Adolph F. Meyer,† It is believed, also, that these data will be of assistance and value to any one who may have occasion to make hydrographic studies of areas within the Great Plateau. The comparisons will be confined to rainfall, stream flow, and mean temperatures (the only data available), and these, necessarily, are more or less incomplete. However, as rainfall and stream flow may be regarded as resultant elements of climate (as distinguished from such causative elements as humidity, wind, sunshine, etc.), the latter are in a sense included in the former—and similarities in rainfalls, stream flows, and mean temperatures are believed in general to show similarities in climates.

Rainfall.—The rainfall relations to be given are based on the rainfalls of 13 years, 1901 to 1913, for which reasonably complete data of observed monthly rainfalls were secured for fourteen rainfall stations in Southern Texas and New Mexico and for fourteen stations in North-

* San Francisco, Cal.

† p. 1932.

TABLE 79.—MEAN, MINIMUM, AND MAXIMUM YEARLY RAINFALLS AT FOURTEEN RAINFALL STATIONS IN SOUTHERN TEXAS AND NEW MEXICO, AND AT FOURTEEN IN NORTHWESTERN MEXICO: FOR THE 13 YEARS, 1901–1913.

TEXAS AND NEW MEXICO.				MEXICO.			
Station.	YEARLY RAINFALLS, IN INCHES.			Station.	YEARLY RAINFALLS, IN INCHES.		
	Mean.	Minimum.	Maximum.		Mean.	Minimum.	Maximum.
San Marcial	9.13	1.17 (1901)	21.79 (1905)	Camargo	6.4	2.4 (1907)	11.8 (1905)
Mesilla Park	9.02	4.02 (1910)	17.09 (1905)	Balleza	18.8	6.0 (1910)	28.3 (1905)
Fort Union	17.24	9.89 (1902)	24.65 (1906)	Guerrero	17.7	5.7 (1910)	25.8 (1905)
Santa Fé	13.88	8.65 (1910)	17.41 (1901)	Parral	15.4	11.6 (1910)	22.7 (1905)
Fort Davis	14.56	8.43 (1910)	23.13 (1905)	Jiminez	11.0	4.0 (1910)	16.4 (1906)
El Paso	9.72	4.03 (1910)	17.80 (1905)	Chihuahua	14.5	5.9 (1901)	21.0 (1906)
Fort Stanton	15.80	9.52 (1903)	22.59 (1905)	Aldama	9.7	7.3 (1909)	13.9 (1905)
Deming	9.31	3.42 (1910)	17.59 (1905)	Ahumada	11.0	3.1 (1903)	20.1 (1911)
Fort Stockton	13.78	4.07 (1910)	20.65 (1905)	Ojinaga	3.2	1.4 (1909)	4.6 (1905)
Fort Clark	20.23	11.62 (1901)	30.94 (1903)	Juarez	9.25	3.8 (1909)	14.8 (1905)
San Antonio	24.94	14.92 (1909)	37.68 (1913)	Ascension	7.1	2.5 (1910)	14.6 (1905)
Fort McIntosh	18.72	4.31 (1901)	36.38 (1903)	Casas Grandes	18.4	4.1 (1910)	35.2 (1905)
Eagle Pass	19.24	8.52 (1910)	30.43 (1913)	La Boquilla	12.0	7.6 (1910)	17.1 (1905)
Roswell	12.91	4.87 (1910)	19.23 (1905)	Pilar de Conchos	15.3	8.0 (1910)	23.9 (1905)
Average	14.90	6.88	24.10	Average	12.18	5.24	19.30

Messrs. Duryea and Haehl.

western Mexico. Such gaps as existed in the rainfall data were filled by careful comparative use of other near-by rainfall records, and the amended yearly rainfalls as used are believed to be reasonably accurate.

A brief summary of the yearly rainfalls at the fourteen rainfall stations in Southern Texas and New Mexico and the fourteen stations in Northwestern Mexico is given in Table 79. The locations of the twenty-eight stations are shown on the isohyetose map, Plate LXIX.

Examinations of Table 79 and of the detailed tables from which it is derived show the following general relations between the rainfalls in Northwestern Mexico and those in Southern Texas and New Mexico:

(*a*) During the 13 years, the maximum yearly rainfall at eight of the fourteen Texas and New Mexico stations and at eleven of the fourteen Mexico stations occurred in 1905; and, during the same year, the rainfalls at each of the six remaining stations in Texas and New Mexico and the three remaining stations in Mexico were above the average and generally high, even though not maxima.

(*b*) During the 13 years, the minimum yearly rainfall at eight of the Texas and New Mexico stations and at eight of the Mexico stations occurred in 1910; and, during the same year, the rainfalls at all but one of the six remaining stations in Texas and New Mexico and at all the six remaining stations in Mexico were below the average and generally low, even though not minima.

The comparative regional yearly rainfalls for each of the 13 years are shown in Table 80.

TABLE 80.—COMPARATIVE REGIONAL YEARLY RAINFALLS IN SOUTHERN TEXAS AND NEW MEXICO, AND IN NORTHWESTERN MEXICO, FOR EACH OF THE 13 YEARS, 1901-13.

Year.	Mean of Fourteen Stations in Texas and New Mexico.		Mean of Fourteen Stations in Mexico.	
	Inches.	Percentage.	Inches.	Percentage.
1901.....	12.8	= 100	9.7	= 75.7
1902.....	13.1	= 100	11.8	= 90.1
1903.....	15.8	= 100	11.3	= 71.5
1904.....	16.4	= 100	13.4	= 81.7
1905.....	23.2	= 100	18.5	= 79.7
1906.....	17.7	= 100	15.3	= 86.5
1907.....	16.2	= 100	11.7	= 72.2
1908.....	12.8	= 100	10.2	= 79.8
1909.....	10.4	= 100	10.3	= 99.1
1910.....	8.5	= 100	6.1	= 71.8
1911.....	16.2	= 100	14.9	= 92.0
1912.....	14.0	= 100	11.1	= 79.3
1913.....	17.1	= 100	13.8	= 80.7
Average.....	14.90	= 100	12.18	= 81.8

From Table 80, the Mexican regional yearly rainfall varies from about 71% to about 99% of that in Southern Texas and New Mexico, averaging about 82% for the 13-year period, and being between 75 and 85% for 7 of the 13 years. Messrs. Duryea and Haehl.

It should be added that, not only do the yearly rainfalls of the two regions resemble each other closely, but that the distributions of the yearly rainfalls among the 12 months also are very similar, there being throughout the two regions a distinct rainy season of about 4 months (June to September, inclusive), during which about three-quarters of the total rainfall of the year occurs.

Also, even such a very unusual vagary as the heavy snowfall (about 6 in.) which occurred at and south of La Boquilla, Mexico, in the latter part of January, 1915, occurred simultaneously at and north of El Paso, Tex.

A more logical comparison of the regional rainfalls of the two areas was made, as follows:

At each of the fourteen Texas and New Mexico stations, each year's rainfall was expressed as a percentage of the mean yearly rainfall at that station for the 13 years; and, for each of the 13 years, a mean was taken of the fourteen values of Texas and New Mexico percentages, to show the general relation throughout Southern Texas and New Mexico of each year's rainfall to the mean rainfall. The yearly rainfalls at each of the fourteen stations in Mexico were treated similarly for each of the same 13 years, and the resulting regional rainfall relations were derived, as shown in Table 81.

In Table 81 the percentage rainfall relations varied from 100% only by from — 12 to + 22% for the separate years, and the means for the 13 years varied by only 0.4%; and after the end of the third year, the maximum difference in the means was only 2.6 per cent.

As it might be that the 13-year period, 1901-13, was too short or too erratic to give true mean yearly rainfalls, that point was investigated for the fourteen rainfall stations of Southern Texas and New Mexico, with the results shown in Table 82.

In Table 82 the mean yearly rainfall at each of the fourteen stations for the 13 years varied from the mean yearly rainfall for the 47 to 19 years by only from — 5.2 to + 15.8%, with an average variation for the fourteen stations (or for the Southern Texas and New Mexico region) of only + 5.2 per cent.

This concludes the comparison of the rainfalls of the two regions, and is believed to show that they have a marked similarity.

Stream Flow.—Some data are available to show the uniformity of the stream flow characteristics throughout Northwestern Mexico, but none is at hand easily to include Texas and New Mexico in the comparison. The available stream flow data are the measured flows of the Rio Conchos for the 14 years, 1900-1913, at its mouth, near

TABLE 81.—REGIONAL YEARLY RAINFALLS IN NORTHWESTERN MEXICO, FOR EACH OF THE 13 YEARS, 1901-13, EXPRESSED AS PERCENTAGES OF THE MEAN RAINFALL FOR THE 13 YEARS, AND COMPARED WITH SIMILAR REGIONAL YEARLY PERCENTAGE RAINFALLS IN SOUTHERN TEXAS AND NEW MEXICO FOR EACH OF THE SAME 13 YEARS.

Year.	YEARLY AND CUMULATIVE RAINFALL PERCENTAGES, COMPARED WITH THE MEAN RAINFALL FOR THE 13 YEARS AS 100 PER CENT.				
	YEARLY PERCENTAGES.		CUMULATIVE AND PROGRESSIVE PERCENTAGES.		
	Texas and New Mexico.	Mexico.	Period, in years.	Texas and New Mexico.	Mexico.
1901.....	86% = 100%	80% = 93%	1	86 + 1 = 86% = 100%	80 + 1 = 80% = 93.0%
1902.....	88% = 100%	97% = 110%	2	174 + 2 = 87.0% = 100%	177 + 2 = 88.5% = 101.7%
1903.....	106% = 100%	93% = 88%	3	280 + 3 = 93.3% = 100%	270 + 3 = 90.0% = 96.5%
1904.....	110% = 100%	110% = 100%	4	390 + 4 = 97.5% = 100%	380 + 4 = 95.0% = 97.5%
1905.....	156% = 100%	152% = 97%	5	546 + 5 = 109.2% = 100%	532 + 5 = 106.4% = 97.5%
1906.....	119% = 100%	126% = 106%	6	665 + 6 = 110.8% = 100%	658 + 6 = 109.7% = 99 0%
1907.....	109% = 100%	96% = 88%	7	774 + 7 = 110.6% = 100%	754 + 7 = 107.7% = 97 4%
1908.....	86% = 100%	84% = 98%	8	860 + 8 = 107.5% = 100%	838 + 8 = 104.7% = 97.4%
1909.....	70% = 100%	85% = 122%	9	930 + 9 = 103.3% = 100%	923 + 9 = 102.6% = 99.4%
1910.....	57% = 100%	50% = 88%	10	987 + 10 = 98.7% = 100%	973 + 10 = 97.3% = 98.6%
1911.....	109% = 100%	122% = 112%	11	1 096 + 11 = 99.8% = 100%	1 095 + 11 = 99.6% = 99 8%
1912.....	94% = 100%	91% = 97%	12	1 190 + 12 = 99.3% = 100%	1 186 + 12 = 98.9% = 99.6%
1913.....	115% = 100%	113% = 98%	13	1 305 + 13 = 100.4% = 100%	1 299 + 13 = 100.0% = 99.6%
Sum.....	(1 305 = 100)	(1 299 = 99.6)			
Mean.....	100 + = 100%	100 − = 99.6%	13	..... 100%	..... 99.6%

Presidio, Tex., and Ojinaga, Mexico, and the estimated flows of the Rio Nazas for the 15 years, 1897-1911, presumably near its mouth and near Torreon, Mexico. Messrs. Duryea and Haehl.

TABLE 82.—MEAN YEARLY RAINFALL OF THE 13 YEARS, 1901-13, AT EACH OF FOURTEEN STATIONS IN SOUTHERN TEXAS AND NEW MEXICO, COMPARED WITH THE MEAN YEARLY RAINFALL AT EACH STATION FOR THE LONGER PERIODS OF 47 TO 19 YEARS, ENDING WITH 1913.

Station.	MEAN YEARLY RAINFALL, IN INCHES.			
	Of the 13 years, 1901-13.	Of the 47 to 19 years ending with 1913.		
San Marcial	9.13 in. = 100%	47 years.	9.25 in.	= 101.3%
Mesilla Park	9.02 in. = 100%	47 years.	8.60 in.	= 95.3%
Fort Union	17.24 in. = 100%	47 years.	18.15 in.	= 105.3%
Santa Fé	13.83 in. = 100%	47 years.	14.01 in.	= 101.3%
Fort Davis	14.56 in. = 100%	47 years.	16.86 in.	= 115.8%
El Paso	9.72 in. = 100%	47 years.	9.20 in.	= 94.8%
Fort Stanton	15.80 in. = 100%	47 years.	16.35 in.	= 103.5%
Deming	9.31 in. = 100%	45 years.	9.64 in.	= 103.5%
Fort Stockton	13.78 in. = 100%	45 years.	15.15 in.	= 110.0%
Fort Clark	20.23 in. = 100%	43 years.	21.24 in.	= 105.0%
San Antonio	24.94 in. = 100%	43 years.	27.94 in.	= 112.0%
Fort McIntosh	18.72 in. = 100%	43 years.	19.64 in.	= 104.8%
Eagle Pass	19.24 in. = 100%	42 years.	21.20 in.	= 110.2%
Roswell	12.91 in. = 100%	19 years.	14.16 in.	= 109.7%
Average	14.90 in. = 100%	47 to 19 years.	15.82 in.	(105.2) = 106.1%

The Rio Conchos flows are the differences in the flows of the Rio Grande above and below the mouth of the Rio Conchos, as measured by the International Boundary Commission, and are believed to be reasonably accurate measures of the flows of that river. No data are easily available, however, for the corresponding Rio Grande drainage areas; hence its flows per square mile from Mexico, Texas, and New Mexico cannot be included in the comparison.

The Rio Nazas data are from the records of a Mexican Government Commission, but are known to be based merely on measurements of cross-sections, slopes, and gauge heights, used in conjunction with assumed Kutter coefficients to estimate stream flows.

The drainage area of the Rio Conchos above its mouth is believed (from probably fairly reliable maps, supplemented by some surveys) to be very nearly 23 000 sq. miles; the Rio Nazas drainage area above the point of its stream gauging is believed to be not far from 13 000 sq. miles.

Because of the inaccuracies of the Rio Nazas flows and drainage area, no comparison can be made of its flows per square mile with those of the Rio Conchos, and all comparisons must be restricted to

Messrs. Duryea and Haehl.

the comparative characteristics of their stream flows, rather than to their comparative flows.

Torreon, near the mouth of the Rio Nazas, is nearly 300 miles south of the mouth of the Rio Conchos.

Comparisons of the stream flow characteristics of these rivers are given in Tables 83 and 84.

TABLE 83.—YEARLY STREAM FLOWS OF THE RIO CONCHOS AND THE RIO NAZAS FOR EACH OF THE 12 YEARS, 1900-11, EXPRESSED AS PERCENTAGES OF THE MEAN YEARLY STREAM FLOW FOR THE 12 YEARS.

Year.	For Each Year.		Cumulative or Progressive Mean.		
	Conchos.	Nazas.	Number of years.	Conchos.	Nazas.
1900.....	74% = 100%	83% = 112%	1	74% = 100%	83% = 112%
1901.....	27% = 100%	34% = 126%	2	50% = 100%	57% = 114%
1902.....	103% = 100%	76% = 74%	3	68% = 100%	65% = 96%
1903.....	46% = 100%	79% = 172%	4	62% = 100%	68% = 110%
1904.....	158% = 100%	84% = 53%	5	81% = 100%	71% = 88%
1905.....	174% = 100%	242% = 139%	6	97% = 100%	100% = 103%
1906.....	198% = 100%	233% = 118%	7	111% = 100%	119% = 107%
1907.....	92% = 100%	53% = 58%	8	109% = 100%	110% = 101%
1908.....	68% = 100%	70% = 103%	9	104% = 100%	106% = 102%
1909.....	95% = 100%	136% = 143%	10	103% = 100%	109% = 106%
1910.....	40% = 100%	32% = 80%	11	98% = 100%	102% = 104%
1911.....	127% = 100%	80% = 63%	12	100% = 100%	100% = 100%
Average.....	100% = 100%	100% = 103.4%	12	.....	.....

As might be expected, even if the flow characteristics of the two rivers were nearly the same (because of the inaccurate method of the Rio Nazas measurements), the Nazas characteristics of the separate years vary rather widely from those of the Conchos, namely from — 47 to + 72%; but the average variation for the 12 years is only + 3.4%; after 3 years, the progressive maximum variations are only — 12 to + 10%; and, after 5 years, the maximum progressive variation is only + 7 per cent.

Not only are the characteristics of the yearly stream flows of the two rivers very similar, but the divisions of the yearly stream flows among the 12 months also are similar as shown by Table 84.

In Table 84 the separate monthly percentages of the year's stream flow vary from only 0.6 to 3.2% for the two rivers, with extremes of — 3.2% (July) to + 2.7% (August); and the cumulative or progressive means have a maximum difference for the two rivers of only 9.7% (for the 7 months, January to July).

It is evident, also, from Table 84, that both rivers have the same well-defined coincident flood season of 3 months (July to September, inclusive), during which the flood flows of the Rio Conchos average

about 67.3% of its year's flow, and the floods of the Rio Nazas about 68.1% of its year's flow. Messrs. Duryea and Haehl.

Mean Temperatures.—That the characteristics of mean monthly temperature are quite uniform throughout Texas and New Mexico, and similar to those at La Boquilla, is shown by a consideration of Fig. 14; and by Table 85, compiled from Plate LXII, Table 37, and later information.

TABLE 84.—CLOSE CORRESPONDENCE BETWEEN THE MONTHLY PERCENTAGE STREAM FLOWS OF THE RIO NAZAS AND THE RIO CONCHOS. MEANS OF THE 12 YEARS, 1900-11.

Month.	For Each Month.			Cumulative or Progressive Mean.			
	Conchos.	Nazas.	Difference.	Number of months	Conchos.	Nazas.	Difference.
January	2.4%	1.6%	— 0.8%	1	2.4%	1.6%	— 0.8%
February	3.0%	1.2%	— 1.8%	2	5.4%	2.8%	— 2.6%
March	1.5%	0.4%	— 1.1%	3	6.9%	3.2%	— 3.7%
April	0.7%	0.1%	— 0.6%	4	7.6%	3.3%	— 4.3%
May	1.0%	0.3%	— 0.7%	5	8.6%	3.6%	— 5.0%
June	2.6%	1.3%	— 1.3%	6	11.4%	4.9%	— 6.5%
July	15.3%	12.1%	— 3.2%	7	26.7%	17.0%	— 9.7%
August	21.3%	24.0%	+ 2.7%	8	48.0%	41.0%	— 7.0%
September	30.7%	32.0%	+ 1.3%	9	78.7%	73.0%	— 5.7%
October	9.5%	12.2%	+ 2.7%	10	88.2%	85.2%	— 3.0%
November	5.8%	8.1%	+ 2.3%	11	94.0%	93.3%	— 0.7%
December	6.0%	6.7%	+ 0.7%	12	100.0%	100.0%	— 0.0%
Year	100%	100%	.....	..	.....	.....	.....

The Botello power-house of Table 85 is within "a long day's horseback ride" of Morelia, capital of the State of Michoacán, Mexico; and has almost the same altitude, climate, and conditions as Morelia.

From a small-scale map, Morelia is almost directly west of the City of Mexico and about 130 miles distant; it is about 540 miles farther south and 260 miles farther east than La Boquilla (at the east end of Lake Conchos), and 600 miles from it in an air line.

Mean temperatures at the Botello power-house were furnished through the courtesy of T. K. Mathewson, M. Am. Soc. C. E., and Messrs. Curtis and Hine, Colorado Springs, Colo., General Managers of the Guanajuato Power and Electric Company. This information will be used hereinafter to check the applicability of Fig. 4 to the quick estimation of an approximate value for the yearly evaporation depth from a reservoir at the Botello power-house, as compared with the *1.72 m., 5.64 ft., or 67.6 in. adopted by Messrs. Curtis and Hine.

* The 1.45 m. given by Mr. Mathewson on p. 1902 was in error, and subsequently was corrected by him to 1.718 m.

TABLE 85.—COMPARATIVE MEAN MONTHLY TEMPERATURES THROUGHOUT TEXAS AND NEW MEXICO, AND AT LA BOQUILLA, MEXICO.

Station.	Elevation above sea level, in feet.	Mean Yearly Temperature, in Degrees Fahrenheit.			Mean Monthly Temperature, in Degrees, Fahrenheit.				
		Temperature.	Departure from average.		July.	January.	Difference.	Departure from average.	
Abilene, Tex	1 740	62.1	.....	— 0.7	83.9 (1887)	38.9 (1888)	= 45.0	+ 9.7	.....
Austin, Tex	600	69.2	+ 6.4	.....	84.2 (1911)	56.5 (1911)	27.7	.....	— 7.6
Brownsville, Tex	50	71.0	+ 8.2	.....	81.2 (1887)	55.0 (1888)	26.2	.....	— 9.1
Corpus Christi, Tex	20	69.4	+ 6.6	.....	82.9 (1887)	48.3 (1888)	34.6	.....	— 0.7
El Paso, Tex	3 762	63.6	+ 0.8	.....	80.9 (1887)	44.8 (1888)	36.1	+ 0.8	.....
" " "	3 762	....	.....	.....	81.0 (1912)	.... (1912)	....	.....	.....
" " "	3 762	61.5	.....	— 1.3	81.5 (1913)	40.0 (1913)	41.5	+ 6.2	.....
Fort Bliss, Tex	3 700	65.3	+ 2.5	.....	83.0 (1889)	48.3 (1890)	34.7	.....	— 0.6
" " "	3 700	64.8	+ 2.0	.....	82.3 (1890)	48.3 (1890)	34.0	.....	— 1.3
Fort Davis, Tex	4 930	59.7	.....	— 3.1	75.4 (1887)	45.0 (1888)	30.4	.....	— 4.9
Galveston, Tex	6	68.7	+ 5.9	.....	81.9 (1887)	49.8 (1888)	32.1	.....	— 3.2
Palestine, Tex	500	64.1	+ 1.3	.....	82.0 (1887)	43.4 (1888)	38.6	+ 3.3	.....
Rio Grande City, Tex	230	71.4	+ 8.6	.....	84.5 (1887)	54.7 (1888)	29.8	.....	— 5.5
San Antonio, Tex	700	66.8	+ 4.0	.....	83.8 (1887)	48.0 (1888)	35.8	+ 0.5	.....
Albuquerque, N. Mex	5 000	56.5	.....	— 6.3	79.1 (1900)	35.4 (1901)	43.7	+ 8.4	.....
" " "	5 000	54.1	.....	— 8.7	75.2 (1903)	36.3 (1903)	38.9	+ 3.6	.....
Carlsbad, N. Mex	3 000	63.1	+ 0.3	.....	80.6 (1899)	42.2 (1899)	38.4	+ 3.1	.....
" " "	3 000	64.1	+ 1.3	.....	80.2 (1901)	44.7 (1901)	35.5	+ 0.2	.....
Elephant Butte, N. Mex	4 250	61.1	.....	— 1.7	81.1 (1909)	42.5 (1910)	38.6	+ 3.3	.....
Fort Stanton, N. Mex	6 150	50.0	.....	— 12.8	65.6 (1887)	36.3 (1888)	29.3	.....	— 6.0
Lake Avalon, N. Mex	3 200	63.8	+ 1.0	.....	82.0 (1909)	46.4 (1910)	35.6	+ 0.3	.....
Santa Fé, N. Mex	7 013	48.9	.....	— 13.9	66.9 (1887)	30.3 (1888)	36.6	+ 1.3	.....
Average (of 20–21)	.....	62.8	(+ 48.9) + 3.8	(— 48.5) — 6.1	80.0	44.7	(35.3) 35.3	(+ 40.7) + 3.4	(— 38.9) — 4.3
‡La Boquilla, Mex	4 300	67.5	+ 0.6	.....	85.0 (1912)	†(51.0) (1912)	(34.0)	+ 5.8	.....
" " "	4 300	65.2	.....	— 1.7	81.0 (1913)	52.0 (1913)	29.0	+ 0.8	.....
" " "	4 300	68.4	+ 1.5	.....	80.9 (1914)	64.3 (1914)	16.6	.....	— 11.6
" " "	4 300	66.5	.....	— 0.4	82.2 (1915)	49.1 (1915)	33.1	+ 5.9	.....
Average (of 4–3)	4 300	66.9	(+ 2.1)	(— 2.1)	82.3	54.1	(28.0) 28.2	(+ 12.5)	(— 11.6)
Botello power-house,* State of Michoacán, Mexico (2 000 m.)	6 560 (1915)	62.4 (1915)	.....	.....	67.8 (1915)	55.2 (1915)	= 12.6	.....	.....
					67.8 (1915)	57.4 (1916)	= 10.4	.....	.....
						Average.....	11.5	.....	.....

* See description following. † Average of Maximum and Minimum (Table 86). ‡ At east end of Lake Conchos.

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In Table 85 the average mean temperatures for July and January at the seventeen stations in Texas and New Mexico are 80.0° Fahr. (July) and 44.5° Fahr. (January); and those at La Boquilla are 82.3 and 50.1°; the average difference between the July and January temperatures is 35.4° at the Texas and New Mexico stations, and 31.3° at La Boquilla; and the departures from the means vary but little. Messrs. Duryea and Haehl.

Table 85 also shows that the variations at La Boquilla between the yearly mean temperatures, the means for January, and those for July were comparatively small during the 4-year period, 1912-15, one year being on the whole much like another; and, that at the Botello powerhouse, though the mean yearly temperature is only about 4° Fahr. lower than that at La Boquilla, the January mean temperature apparently is about 14° lower; and the excess of the July over the January mean temperature apparently is about 17° less than at La Boquilla.

It should be noted that some of the stations in Table 85 are outside the limits of the "Great Plateau".

Table 85 contains only yearly mean temperatures and those for July and January. Throughout the Plateau portion of Texas, New Mexico, and Arizona the nights usually are much colder than the days; and that the same condition holds at La Boquilla also is shown by Table 86, which gives the averages of the daily readings throughout each month by a maximum and minimum thermometer. Table 86 also defines more closely the temperature conditions at La Boquilla.

From the foregoing data of rainfalls, stream flows, and mean temperatures it is apparent that, at least in those respects, the climatological conditions are quite uniform throughout Southern Texas and New Mexico and Northwestern Mexico; and this uniformity was even more apparent from diagrams than from tables.

It is regretted that data relating to humidity, sunshine, wind, etc., were not secured at Lake Conchos, as suggested by Mr. Meyer, but this was prevented by the extreme difficulties of instituting any kind of new work (even observations) in Mexico at that time, and by the need of quick results in a practical engineering problem.

Referring to some criticisms, the investigation was not intended to be, nor was it considered, a "scientific" one, but was frankly empirical, making use of such observed evaporation depths in Southern Texas and New Mexico as were of record to estimate (by empirical modifications) the probable yearly evaporation depth in Northern Mexico. However, the writers do not admit in any spirit of apology that their work was empirical. On the contrary, they believe that "scientific" methods of investigating evaporation (such as that of Professor Bigelow, which was an investigation of the law of evaporation rather than of its amount) are of comparatively little value in practical engineering problems, and this view seems to be Mr. Grunsky's also.

TABLE 85.—COMPARATIVE MEAN MONTHLY TEMPERATURES THROUGHOUT TEXAS AND NEW MEXICO, AND AT LA BOQUILLA, MEXICO.

TABLE 86.—AVERAGE MAXIMUM AND MINIMUM TEMPERATURES AT LA BOQUILLA, MEX., FOR EACH MONTH; BEING THE AVERAGES OF THE DAILY MAXIMUM TEMPERATURES ON A MAXIMUM THERMOMETER, AND OF THE DAILY MINIMUM TEMPERATURES ON A MINIMUM THERMOMETER.

Month.	Year 1912.			Year 1913.			Year 1914.			Year 1915.			Average for 4 Years.			
													Average of Columns.			
	Maximum. (1)	Minimum. (2)	Difference. (3)	Maximum. (4)	Minimum. (5)	Difference. (6)	Maximum. (7)	Minimum. (8)	Difference. (9)	Maximum. (10)	Minimum. (11)	Difference. (12)	(1), (4), (7), and (10). Maximum. (13)	(2), (5), (8), and (11). Minimum. (14)	(3), (6), (9), and (12). Difference. (15)	(13) and (14). Mean. (16)
January	*(66°)	†(36°)	‡(30°)	67°	37°	30°	69°	(36°)	(33°)	63°	§35°	28°	66.2°	36.0°	30.2°	51.1°
February	(70)	(41)	(29)	68	45	23	73	40	33	70	39	31	70.3	41.8	29.0	55.8
March	(74)	(45)	(29)	76	51	25	74	43	31	71	41	30	73.7	45.0	28.7	59.3
April	85	(56)	(29)	82	61	21	89	55	34	85	52	33	85.2	56.0	29.2	70.6
May	95	(62)	(33)	93	66	27	96	62	34	96	58	38	95.0	62.0	33.0	78.5
June	97	(65)	(33)	93	65	28	96	66	30	‖104.5	65	40	97.6	65.2	32.7	81.4
July	98	(66)	(30)	98	70	28	96	66	30	¶101.3	63	38	97.1	66.3	30.3	81.7
August	92	67	25	89	65	24	94	66	28	92	56	36	91.7	63.5	28.2	77.6
September	88	62	26	79	57	22	87	61	26	90	55	35	86.0	58.7	27.3	72.3
October	81	54	27	73	(51)	(33)	80	54	26	90	44	46	81.0	50.7	33.0	65.8
November	69	41	28	65	(41)	(34)	70	47	23	85	35	50	72.2	41.0	33.7	56.6
December	64	39	25	58	(34)	(34)	64	36	28	77	**28	49	65.7	34.5	34.0	50.1
(Average)	.....	.....	(28.7)	.....	.....	(27.0)	.....	.....	(29.7)	.....	.....	(37.8)	.....	.....	.....	.....
Year	81.6°	52.8°	28.8°	78.0°	53.6	24.4°	82.3°	52.7°	29.6°	85.4°	47.6°	37.8°	81.8°	51.7°	30.8°	66.8°

All values in parentheses are for times during which the corresponding thermometers were broken or out of order.

* Mean of Columns (4), (7), and (10).
† Mean of Columns (5), (8), and (11).
‡ Mean of Columns (6), (9), and (12), etc.

Minimum and Maximum Daily Temperatures, in Degrees, Fahrenheit:
§ Minimum—January, 1915; 18th = +21°, 19th-21st = +23° (heavy snowstorm).
‖ Maximum—June, 1915; 18th and 27th = 108°.
¶ July, 1915; 1st = 111°.
** Minimum—December, 1915; 22d and 25th = +19°, 29th = +18°.

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As to empirical methods, large and important parts of the field of engineering are dependent almost wholly on empiricism. One of the most familiar examples is the flow of water, the useful knowledge of which has been almost wholly empirical, ever since the first appearance of the empirical Chezy formula in 1775.

The practical engineering problems before the writers in the investigation of the Rio Conchos or La Boquilla project were as follows:

A.—The estimation of the safe water supply and power production of the project;

B.—Reports on the safety of the foundations of each of the three dams; on the safety of the plans and of the past construction; and on plans and cost for the completion of the project;

C.—A preliminary report on the irrigation opportunities.

Also, the time for the investigation was somewhat limited by the fact that it was desired to resume as soon as practicable the then abandoned construction work, and push it to completion as rapidly as possible, and because the results of the new investigations were necessary as precedents to the re-financing of the project, the former financial arrangements having been disturbed by the effects of the war in Mexico which caused the temporary abandonment of the construction in September, 1913.

The determination of a reasonably safe value for the yearly evaporation depth, even though more important than usual, was, after all, only a minor factor in *A*. The evaporation depth had to be made use of early in the investigation of *A*, however, for the following reason:

The stream flow data of the Rio Conchos consisted of 14 years (1900-13) measured flows at its mouth (its junction with the Rio Grande), and of 4 years (1910-13) measured flows at La Boquilla. However, the flows of 1912 and 1913 at La Boquilla were measured largely by their being impounded in the surveyed reservoir of Lake Conchos, and hence were exclusive (as impounded) of the large evaporation losses from the lake. The flows of 1910-13 were necessary as data to establish a safe proportion between the flow of the Rio Conchos at La Boquilla and that at its mouth; hence the evaporation depth, as estimated and checked up to the middle of February, 1914, had to be made use of at once, in order not to delay the more important and more protracted investigation of the safe water supply and power production. As a precaution, the observations of pan evaporation at Lake Conchos were continued until June, 1914, when preliminary statements of conclusions were rendered to the Company; and the final reports were completed in October and November, 1914.

The pan measurements of evaporation were continued at Lake Conchos for several months after June, 1914, but this was not learned by the writers until December, 1915. Efforts were made to secure the

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notes of the later pan measurements for use in this closure, but the La Boquilla project (which had been operating commercially since September 29th, 1915, selling some of its electric power to mines at Parral) was again abandoned in January, 1916, before the notes were secured. The furnishing of power to Parral was resumed on July 25th, 1916, but the notes of the additional pan evaporations have not yet been secured.

The various discussions on the paper will now be considered. In answering them the writers will group together all comments on the same subject, as far as practicable.

Salton Sea Evaporation Pans Circular, and not Square.—As stated by Messrs. C. E. Grunsky,* Charles W. Comstock,† and Charles H. Lee,‡ the evaporation pans used by Professor Bigelow in his Salton Sea experiments were circular, instead of square, as erroneously assumed by the writers. In making their investigation of the evaporation from Lake Conchos, and even later in preparing the paper, the writers had access only to a summary of the Salton Sea experiments, giving the conclusions reached, but describing only inadequately the methods and apparatus. From that information it was assumed that the Bigelow experiments were carried out in accordance with usual practical engineering methods, and with usual standard apparatus—and hence that the pans were square.

However, while acknowledging their error in assuming the Salton Sea evaporation pans to have been square, when in fact they were circular, it is hard to believe that any material error can arise therefrom, and this view apparently is Mr. Comstock's also. Although the evaporation depth from a pan 3 ft. square may be slightly different from that from a pan 3 ft. in diameter, it seems very unlikely that (from a practical engineering standpoint) any material error can result from applying the relative observed evaporation depths from a series of pans 2, 3, 4, etc., ft. in diameter, to a series of pans 2, 3, 4, etc., ft. square.

Subsidiary Conclusion (a) of Table 2.—This conclusion (of Professor Bigelow, but accepted and used by the writers) gives the presumed relative evaporation depths from a series of 2-ft. to 6-ft. pans, all the conditions of climate, exposure, etc., supposedly having been identical, and the evaporation conditions varying only in the sizes of the pans.

As stated already, in discussing square *versus* circular pans, it was assumed by the writers (incorrectly) that the Salton Sea evaporation experiments of Professor Bigelow were made on practical engineering lines; and indeed that seems to have been the earnest desire of Mr.

* pp. 1969–71.
† pp.1949–50
‡ p. 1925.

Grunsky, a member of the Conference Board which outlined a programme for the work. Messrs. Duryea and Haehl.

It is apparent, however, from information given in Mr. Grunsky's discussion,* that the "floating-pans" were not floating in the usual sense; but that, instead (with a depth of pan of 10 in. and of contained water of presumably about 8 to 9 in.), they were immersed in the water of a slough only about 3 or 4 in.

However, in investigating the effect of size of pan on evaporation depth, Professor Bigelow used three series of pans:

(1) Two pans of different sizes alongside each other, one perhaps on a platform and the other on the ground, but presumably not banked by earth, as is usual with "land-pans";
(2) Three pans of different sizes on a tower platform, half a mile from shore, and as close to the water surface as was practicable; and
(3) Four pans of different sizes, supported by adjoining rafts so that the pans were immersed only 3 or 4 in. in the water of a slough;

and, of these three series of tests, Professor Bigelow states "the ratios [of evaporation depth, from pans of different sizes] are quite steady and the results have been incorporated into the final values of the coefficient, C_2".

The full range of practical conditions for an engineering investigation of pan evaporation, of course, should have had as extremes "land-pans" embedded in or banked around with earth, and "floating-pans" floating free and thus fully immersed in water; and it is much to be regretted that Professor Bigelow's tests did not include such extreme conditions. However, his Series (1) approximated the land-pan condition, his Series (3) the floating-pan condition, and his Series (2) was in a condition intermediate between (1) and (3). Thus, out of five conditions, he investigated three, throughout which range the ratios were "quite steady"—presumably meaning by this nearly constant. Hence, if the results of the three series which were investigated are plotted on a diagram, with the "conditions" as abscissas and the values of C_2 as ordinates, it would not be an unusual assumption, or one which is not often resorted to in engineering investigations, if the approximately horizontal line representing the value of C_2, throughout the range of the three conditions which were investigated, should be extended or projected—on the one hand to include the full "land-pan" conditions, and on the other to include the full conditions of "floating-pans".

It is much to be regretted that the two conditions of banked land-pans and full floating-pans were not included by Professor

* pp. 1969–70.

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Bigelow in his investigations, especially as only those two conditions are of direct value in practical engineering investigations of evaporation. Nevertheless, it seems to the writers that the conclusions embodied in (*a*) of Table 2 still presumably are true to within a fair degree of approximation, even when applied to "land-pans" and to "floating-pans"; and that (until and unless proven so to be untrue by further and direct investigation) the values of (*a*) may be used to modify evaporation depths measured in land- or floating-pans of different sizes to the presumptive depths, if pans of a uniform size had been used, without fear of thus introducing material errors into the operation and into its final results.

At the very least, it is believed that comparisons between evaporation depths from pans of different sizes thus modified for sizes of pans, will show relations much nearer the true ones than if the depths are not so modified. The writers are not aware of any other or better established relations between evaporation depths from pans of different sizes than those advanced by Professor Bigelow, and none has been pointed out by any of those who have discussed the paper. It is admitted that these values should not be accepted as final or conclusive for the usual conditions of land-pans and floating-pans, and it is not thought that they were thus considered, even by Professor Bigelow. However, until further tests and the establishment of more reliable values, it is believed that the value and truth of combinations of evaporation depths observed in pans of different sizes will be increased materially by the use of Professor Bigelow's values to modify (before combination) the observed depths to a uniform size of pan.

Mr. Grunsky's statement,* that the values of Subsidiary Conclusion (*a*), Table 2, apply only to a factor in the formula for the evaporation depth, and not to the evaporation depth itself, is of course true. However, the other factors in the formula apparently are almost independent of the size of the pan, and the variable, C_2, is apparently of preponderating influence on the evaporation depth; hence it does not seem that the assumption that the relative evaporation depths vary as do the values of C_2 should lead to material error.

Mr. Grunsky's formula,

$$C_2 = 0.023 + 0.010\frac{p}{a},$$

for land-pans 10 in. deep, not embedded in the soil, in a climate comparable with that of the Salton Sea, is of much interest as expressing the variable value of C_2 in terms only of the size of pan, and with values in practical agreement with those of Professor Bigelow.

*p. 1970.

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Mr. Comstock gives values of C_2* as Professor Bigelow's which differ materially from those given in that writer's "Abstract of Data No. 4", and repeated in Table 7. When compared for relative values, however (the only values of interest in connection with Subsidiary Conclusion (*a*)), it is found that for 4-ft. and 6-ft. pans the relative values of C_2 are almost identical for the two sets—though this close correspondence does not hold for the 2-ft. pan.

Mr. Comstock's statement, that the higher water temperatures and greater evaporation depths, observed at Indio in 1907 to exist in smaller than in larger pans, are to be expected in land-pans but cannot exist in floating-pans, does not appear to the writers to be justified. It is presumable that the higher temperatures are due to a continuous (during sunshine) accession of heat to the small confined body of water from sun heat on the metal rim and bottom of the pan, such heating effect evidently being greater in smaller than in larger pans; and it is to be expected that floating- as well as land-pans will be thus affected, though perhaps to a less degree.

As stated by Mr. Comstock, Professor Bigelow's explanation of the reason that evaporation depths are greater from smaller than from larger pans (because of the more effective clearing of the vapor blanket from the smaller pans by wind action) does not seem convincing as accounting for the sole or even the most important cause. It is believed that a much more important cause is that suggested by Mr. Grunsky: the evaporation (by sun heat on the metal rim of the pan and by wind action) of the thin film of water drawn up on the inside of the pan by the capillary action of the pan surface. It is evident that the evaporation depth from this cause must increase as the size of pan decreases.

Also, it is not unlikely that other causes, as yet unrecognized, contribute to make the evaporation depths from small pans greater than those from larger ones.

Subsidiary Conclusion (*b*) *of Table 2.*—This conclusion is the writers', not one of Professor Bigelow's, and is that the evaporation depth from a "floating-pan" is about 80% of that from a near-by "land-pan" of the same size.

Conclusion (*b*) is discussed by Messrs. Grunsky,† Lee,‡ Comstock,§ Post,‖ Ledoux,¶ and Hawgood;** and the various values advanced for it are given in Table 87.

The Kingsburg evaporation depths of Mr. Grunsky were measured in pans 3 ft. square and 15 in. deep, with the water in the pans 10 in.

* p. 1951.
† pp. 1975–78.
‡ p. 1925.
§ pp. 1951–52.
‖ pp. 1906–07, 1917.
¶ pp. 1998–2000.
** pp. 1946–48.

TABLE 87.—VARIOUS VALUES ADVANCED FOR THE PROPORTION WHICH THE EVAPORATION DEPTH FROM A FLOATING-PAN BEARS TO THAT FROM A NEAR-BY SIMILAR LAND-PAN OF THE SAME SIZE.

(For the full cycle of a year.)

Authority.	Locality.	Period and dates.	No. of Pans.		Evaporation depth from floating-pans, as a percentage of that from land-pans.
			Floating.	Land.	
C. E. Grunsky	Kingsburg, Cal.	4 years, 1881-85	1	1	78%
C. H. Lee	Owens River, Cal.	1 year, 1910-11	1	1	75%
C. W. Comstock	Coyote, Cal., and Granite Reef, Ariz.	2 years, Coyote, and 1 year Granite Reef	3	4	82%
Mean	Arid regions	6½ years	5	6	78%
W. S. Post	San Diego, Cal.	33 months, January, 1913-September, 1915.	6	4	99%
W. S. Post	San Diego, Cal.	5 months, May-September, 1915	1	1	98%
J. W. Ledoux	Allegheny Mountains, Pa.	Several months, 1888-89	..	..	*100%
H. Hawgood	.....	.....	..	..	No fixed relation
Duryea and Haehl	Coyote, Cal.	1 year, 1904	2	3	72.8%
Duryea and Haehl	Coyote, Cal.	1 year, 1905	2	3	85.0%
Mean	Coyote, Cal.	2 years, 1904-05	2	3	78.9%

* "Land gages give the same results as floating gages, provided the temperature of the water in the evaporating vessel is kept the same as that of the water in the reservoir".

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or less in depth. The floating-pan was floating in the Kings River. The land-pan was on a railroad trestle for the first 3 months, exposed on all sides to sun and air, and after that on land, set a few inches in the ground, and with earth embanked around it to the elevation of its water surface. Messrs. Duryea and Haehl.

The Owens River evaporation depths of Mr. Lee were measured in pans 3 ft. square and 10 in. deep, with the water in the pans 8 in. or less in depth. The land-pan was embedded in the ground, and the floating-pan floated with from 1 to 5 ft. depth of water beneath it, depending on the stage of the river.

Mr. Comstock's value, 82%, is one derived from Tables 8 and 11, by rejecting the values for Salton Sea and California, Ohio, and combining those for Coyote (24 months), Granite Reef (presumably 12 months), and Lake Conchos (5 months, Table 11), with a resultant weighted mean of 81.5 per cent. If it is worth while to weight the different values, it would seem more logical to give the Coyote value double the weight corresponding to its 24 months (because of its two floating-pans and three land-pans) and to include also the 2-months Lake Conchos value of Table 10. The weighted value would not have been changed materially thereby, however, it being then 81.8 per cent.

The San Diego evaporation depths of Mr. Post were obtained in pans 3 ft. square and 18 in. deep, with the water in the pans presumably about 16 in. or less in depth. The pans giving his second value (for 5 months only) are included with those giving his value for the 33 months.

The Coyote evaporation depths of the writers were measured in pans 3 ft. square and 12 in. deep, with the water in the pans about 10 in. or somewhat less in depth. The land-pans were embedded in the ground and banked around by earth nearly to their rims; and the floating-pans floated on the Laguna Seca with from 3 to 6 ft. of water beneath them, depending on the stage of the lagoon.

For the full cycle of a year, all the authorities of Table 87, except Messrs. Post, Ledoux, and Hawgood, are in practical agreement with the value of "about 80%" adopted by writers. As values in the field of hydrography, rainfall, etc., Mr. Comstock's results, 83.4% (unweighted) and 82% (weighted), are regarded by the writers as satisfactory agreements with the adopted 80% value, rather than as material disagreements.

The values of Messrs. Post, Ledoux, and Hawgood (and of Table 87 in general), will be discussed hereinafter, in connection with the consideration of Table 88—data of the relative observed evaporation depths from land-pans and from floating-pans, for periods shorter than the full cycle of a year. The evaporation tests of Table 88 are those by Messrs. Grunsky and Lee, and by the writers in Table 87 in monthly detail.

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TABLE 88.—DATA OF EVAPORATION DEPTHS FROM FLOATING-PANS, SIZE (ALL PANS 3 FT. SQUARE), FOR EACH MONTH OF THE

	Separate Months									Cumu			
	Coyote, Cal.			Kingsburg, Cal.					Owens River, Cal.	Total period, in months.	Coyote, Cal.		
Months.	Year 1904.	Year 1905.	2 Years Mean.	Year 1882.	Year 1883.	Year 1884.	Year 1885.	4 Years Mean.	1 Year Mean.		Year 1904.	Year 1905.	2 Years Mean.
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
	%	%	%	%	%	%	%	%	%		%	%	%
Jan.....	67.8	68.3	68.0	45.0	150.0	140.0	100.0	80.8	*76.7	1	67.8	68.3	68.0
Feb.....	48.1	104.0	76.0	109.5	142.8	83.3	107.7	109.0	*111.2	2	58 6	85.9	72.1
Mar.....	69.6	99.3	84.4	67.9	98.4	81.0	109.0	90.5	70.4	3	63.6	91.9	77.7
Apr.....	96.8	115.6	106.0	59.8	103.8	88 9	59.2	74.8	74.4	4	75.9	100.6	88.2
May.....	61.2	71.3	66.2	36.5	51.6	80.0	51.6	51.1	78.1	5	68,9	90.0	79.4
June.....	72.9	64.5	68.7	50.5	58.8	58.4	81.5	62.1	83.0	6	70.3	80 9	75.6
July.....	73.2	72.2	72.7	61.4	78.3	56.8	79.0	69.5	67.7	7	71.1	78.1	74.6
Aug.....	70.2	95.0	82.6	76.1	94.0	55.2	100.0	83.9	69.3	8	70.9	81.7	76.3
Sept.....	84.3	100.4	92.3	82.7	108.6	61.6	97.0	87.8	71.7	9	72.6	84.5	78.6
Oct.....	65.2	82.7	74.0	59.2	138.0	112.8	130.6	117.3	75.0	10	72.1	84.3	78.2
Nov	91.7	95.0	93.4	115.0	121.5	166.7	†101.5	101.7	83.1	11	72.7	84.9	78.8
Dec.....	76.1	94.0	85.0	94.5	133.3	128.5	†95.2	94.1	77.0	12	72.8	85.0	78.9
(Average) Year	72.8	85.0	78.9	‡67.4	‡89.5	‡77.2	‡91.5	77.7	75.2	Year.	72.8	85.0	78.9

* January and February of 1911, March to December of 1910.

† Values for November and December, 1885, equal mean values for those months

‡ Values actually observed during the 4 years were as follows:

November, 1881 to October, 1882 = 61.0%
" 1882 " " 1883 = 88.8
" 1883 " " 1884 = 72.7
" 1884 " " 1885 = 89.9 mean of the 4 years = 78.1 per cent.

§ Columns (3), (8), and (9) and (13), (18), and (19) are "weighted" for
Columns (3) and (13)—2 pairs of pans for 2 years = 4 pan (pair)-years.
Columns (8) and (18)—1 pair of pans for 4 years = 4 pan (pair)-years.
Columns (9) and (19)—1 pair of pans for 1 year = 1 pan (pair)-years.
"Weight" of Columns (22) and (23) = 9 pan (pair)-years.

From Table 87 it is apparent that Mr. Post's conclusion from his San Diego experiments (that yearly evaporation depths from floating-pans are about the same as those from land-pans, both for a full cycle of a year and for the summer season only), Mr. Ledoux' conclusion to the same effect, and Mr. Hawgood's conclusion (that the relation between the evaporation depths from land- and floating-pans is a variable one, depending on many other influences than the floating and land conditions, and not expressible by a practically constant percentage)—all three are in disagreement with all the other data in Table 87 for this relation.

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LATIVE OR PROGRESSIVE PERIODS.						MEANS OF SEPARATE MONTHS AND PROGRESSIVE PERIODS.			
Kingsburg, Cal.					Owens River.	Unweighted means of Columns:		§ Weighted means of Columns:	
Year 1882.	Year 1883.	Year 1884.	Year 1885.	4 Years Mean.	* 1 Year Mean.	(3), (8), and (9). (Separate months).	(13), (18), and (19). (Progressive periods).	(3), (8), and (9). (Separate months).	(13), (18), and (19). (Progressive periods).
(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)
%	%	%	%	%	%	%	%	%	%
45.0	150.0	140.0	100.0	80.8	*76.7	75.2	75.2	74.7	74.7
67.2	145.5	136.0	107.1	96.7	*94.8	98.7	87.9	94 6	85.6
67.6	110.7	102.2	108.3	93.2	80.0	81.8	83.6	85.6	84.8
64.2	108.1	96.5	87.3	85.9	77.4	85.1	83.8	88.6	86.0
51.6	90.5	88.4	68.9	70.4	77.7	65.1	75.8	60.8	75.2
51.8	75.9	77.0	73.8	67.2	79.1	71.3	74.0	67.4	72.3
54.1	76.7	70.2	75.4	67.9	76.6	70.0	73.0	70.7	71.8
58.2	81.4	65.7	81.2	71.5	75.3	78.6	74.4	81.7	74.1
60.8	84.9	65.6	83.5	73.8	75.0	83.9	75.8	88.0	76.1
61.1	88.2	69.8	87.0	76.4	75.0	88.7	76.5	93.4	77.0
66.9	89.0	75.7	†91.3	77.4	75.3	92.7	77.2	95.9	77.8
67.4	89.5	77.2	†91.5	77.7	75.2	85.4	77.3	88.2	76.8
.....	.....	.....	.....	.....	.....	(81.4)	.....	(82.5)	.....
67.4	89.5	77.2	91.5	77.7	75.2	77.3	77.3	76.8	76.8

for the 4 years, November, 1881, to October, 1885, inclusive.

Columns (22) and (23) as follows:

Because of the fairly full data of Table 87 to the contrary (derived from experiments at three widely separated places and extending over periods of from 1 to 4 years), it is believed that Mr. Post's San Diego conclusion is not true generally elsewhere, but was due to the influences of some unknown and unrealized local differences of climate and exposure, as between his land-pans and his floating-pans; and that this explanation applies also to Mr. Ledoux' conclusion.

And, notwithstanding Mr. Hawgood's conclusion, it is still believed that (with the land-pans embedded in the ground or embanked around with earth, and with the floating- and land-pans identical in size,

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material, etc., and with identical conditions of exposure, etc.) there is in general (at least for full yearly cycles) a fairly constant relation between evaporation depths from land-pans and those from floating-pans, and that the evaporation depths from floating-pans are about 80% (at least for full yearly cycles) of those from land-pans of the same size, as adopted for Subsidiary Conclusion (*b*) by the writers.

Except Columns (3), (13), and (20) to (23), all the columns of Table 88 were derived by dividing the inches of depth of evaporation in the floating-pan by the inches of depth in the land-pan, for the corresponding single months or for the longer continuous periods of from 2 to 12 months; but Columns (3), (13), and (20) to (23) are merely means of the percentages given in certain of the other columns. Hence (except for the cumulative or progressive columns), the average of the twelve percentage values in each column will not check exactly the "year" percentage value at the bottom of the table. However, for Column (20) the average is only $(81.4 \div 77.3) = 105.2\%$ of the "year", and for Column (22) only $(82.5 \div 76.8) = 107.5\%$, and for three other columns, tried as rough checks, the differences are still less.

In Table 88 all the cumulative or progressive periods begin with January, and it is evident that different sets of progressive values would be obtained by beginning with February, March, etc., there being thus twelve sets of values, in all of which only those for 12 months would be identical.

However, the working out of such twelve sets of values would be too onerous and voluminous for any present use, and it is believed that sets of values for progressive periods beginning with other months than January can be worked out quickly and with sufficient accuracy by combinations of the percentages of Column (22) for the separate months.

The only progressive periods of special interest in connection with the Lake Conchos investigation are the following:

12 months (beginning at any month).
The 12 separate months.

Used by Mr. Post:
- 3 months, December to February;
- 4 " March to June;
- 5 " July to November.

Used by the writers for Lake Conchos:
- 2 months, (about) middle of January to early in March;
- 5 " (about) " " " " " " June;
- 4 " October to January (lake observations);
- 7 " October to April (" ").

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TABLE 89.—COMPARISON OF VARIOUS VALUES OF THE PROPORTION WHICH THE EVAPORATION DEPTH FROM A FLOATING-PAN BEARS TO THAT FROM A LAND-PAN, FOR VARIOUS PROGRESSIVE PERIODS LESS THAN 12 MONTHS.

Progressive periods and separate months.	PERCENTAGE, INCHES IN FLOATING-PAN ÷ INCHES IN LAND-PAN.					
	Computed from Column (22) of Table 88.		Values advanced by Mr. Post.		Uniform value adopted by writers.	
	(1)	(2)	(3)	(4)	(5)	(6)
12 months (any consecutive 12)	76.8%	= 100%	99%	= 128.8%	80%	= 104.2%
Separate Months:						
January	74.7%	= 100%	150 ±%	= 193%	80%	= 115.2%
February	94.6%	= 100%	150 ±%	= 152%	80%	= 91.0%
March	85.6%	= 100%	108 ±%	= 121%	80%	= 100.4%
April	88.6%	= 100%	108 ±%	= 117%	80%	= 97.1%
May	60.8%	= 100%	108 ±%	= 171%	80%	= 141.4%
June	67.4%	= 100%	108 ±%	= 154%	80%	= 127.5%
July	70.7%	= 100%	90 ±%	= 122%	80%	= 121.6%
August	81.7%	= 100%	90 ±%	= 100%	80%	= 105.2%
September	88.0%	= 100%	90 ±%	= 98%	80%	= 97.8%
October	93.4%	= 100%	90 ±%	= 93%	80%	= 92.1%
November	95.9%	= 100%	90 ±%	= 90%	80%	= 89.8%
December	88.2%	= 100%	150 ±%	= 164%	80%	= 97.5%
(Average)	(82.5 = 107.5%)	(100%)	(111 = 112%)	(131.8%)	(80%)	(106.4%)
Year	76.8 = 100%	100%	99 = 100%	= 128.8%	80%	= 104.2%
December to February, inclusive (3 months—Post at San Diego)	79.8%	= 100%	134.0%	= 168.0%	80%	= 100.3%
March to June, inclusive (4 months—Post at San Diego)	70.4%	= 100%	96.5%	= 137.0%	80%	= 113.7%
July to November, inclusive (5 months—Post at San Diego)	80.0%	= 100%	80.4%	= 104.5%	80%	= 100.0%
January to March (*2 months—writers at Lake Conchos)	81.8%	= 100%	127.8%	= 156.0%	80%	= 97.9%
January to June (†5 months—writers at Lake Conchos)	75.8%	= 100%	97.4%	= 128.5%	80%	= 105.6%
October to January, inclusive (4 months—writers at Lake Conchos)	81.8%	= 100%	112.2%	= 137.0%	80%	= 97.9%
October to April, inclusive (7 months—writers at Lake Conchos)	82.5%	= 100%	108.0%	= 131.0%	80%	= 97.1%

* 15 days in January plus all of February plus 9 days in March.

† 15 days in January plus all of February to May plus 3 days in June.

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Progressive values for the various periods above are given in Table 89, as worked out from Column (22) of Table 88, as advanced by Mr. Post, and as adopted by the writers at the uniform value of 80 per cent. In working out periods including only parts of months, the monthly values will be weighted in proportion to the fraction of the whole month covered by the observations.

Column (1) of Table 89 is largely copied directly from Column (22) of Table 88, as the most reliable values. The values of Column (3) of Table 89 are from Mr. Post's discussion.* It should be noted that the average of the twelve values for the separate months, is 107.5% of the "year" for Column (1) and 112% for Column (3). Hence, in deriving the comparative percentages of Columns (2), (4), and (6) from combinations of Columns (1), (3), and (5), an attempt was made to correct roughly for such lacks of check, as shown in the examples which follow. It is believed that (even though still only approximate) the values of Columns (2), (4), and (6) are nearer truly comparative than if such corrections had been omitted.

Examples.—

Columns (4) and (6), for Separate Months:

$$\text{Column (4)} = \frac{\text{Column (3)}}{1.12} \div \frac{\text{Column (1)}}{1.075} = \frac{(1.075)}{(1.12)} = 0.96$$

$\times$ Column (3) $\div$ Column (1) $= 0.96 \times 150 \div 74.7 = 193\%$ for January, etc.

$$\text{Column (6)} = \frac{\text{Column (5)}}{1.00} = \frac{\text{Column (1)}}{1.075} = 1.075 \times \text{Column (5)}$$

$\div$ Column (1) $= 1.075 \times 80 \div 74.7 = 115.2\%$ for January, etc.

Columns (4) and (6), for Progressive Periods:

Column (4) = Column (3) $\div$ Column (1); as $134.0 \div 79.8 = 168\%$ for December to February, etc.

Column (6) = Column (5) $\div$ Column (1); as $80 \div 79.8 = 100.3\%$ for December to February, etc.

Columns (1) and (3), for Progressive Periods:

Column (1)—(Dec. to Feb.).		Column (3)—(Dec. to Feb.).	
Dec.	88.2	Dec.	150.
Jan.	74.7	Jan.	150.
Feb.	94.6	Feb.	150.
Mean	$= 85.8 \div 1.075 = 79.8\%$, etc.	Mean	$= 150. \div 1.12 = 134.0\%$, etc.

*pp. 1906-07.

Column (1)—Jan. to Mar., 2 months (nearly).	Column (3)—Jan. to Mar., 2 months (nearly). Messrs. Duryea and Haehl.
Jan. (15 days) $74.7 \times 0.48 = 36.2$	Jan. $150. \times 0.48 = 72.$
Feb. (1 month) $94.6 \times 1.00 = 94.6$	Feb. $150. \times 1.00 = 150.$
Mar. (9 days) $85.6 \times 0.29 = 24.8$	Mar. $108. \times 0.29 = 31.3$
1.77 months $= 155.6$	1.77 months $= 253.3$
Mean for period $= 88.0$	Mean for period $= 143.2$
$88.0 \div 1.075 = 81.8\%$	$143.2 \div 1.12 = 127.8\%$
Column (1)—Jan. to June, 5 months (nearly).	Column (3)—Jan. to June, 5 months (nearly).
Jan. (15 days) $74.7 \times 0.48 = 36.2$	Jan. $150. \times 0.48 = 72.$
Feb. (1 month) $94.6 \times 1.00 = 94.6$	Feb. $150. \times 1.00 = 150.$
Mar. (1 month) $85.6 \times 1.00 = 85.6$	Mar. $108. \times 1.00 = 108.$
Apr. (1 month) $88.6 \times 1.00 = 88.6$	Apr. $108. \times 1.00 = 108.$
May (1 month) $60.8 \times 1.00 = 60.8$	May $108. \times 1.00 = 108.$
June (3 days) $67.4 \times 0.10 = 6.7$	June $108. \times 0.10 = 10.8$
4.58 months $= 372.5$	4.58 months $= 556.8$
Mean for period $= 81.4$	Mean for period $= 109.2$
$81.4 \div 1.075 = 75.8\%$, etc.	$109.2 \div 1.12 = 97.4\%$, etc.

It should be noted that in Table 89 the ("Average") and the "Year" (below the Separate Months of both Column (4) and Column (6)) agree quite closely with one another.

The discussion of Subsidiary Conclusion (*b*) now can be completed by the help of Tables 88 and 89.

Column (1) of Table 89 is believed to present fairly well-established values of (*b*), with the weight of 9 pan (pair) years of actual evaporation observations made by Messrs. Grunsky, Lee, and the writers, in three widely separated localities; and all made with 3-ft. square pans and with floating-pans and land-pans all placed in a usual or standard manner. It is not claimed that the values of Column (1) cannot be amended and improved by the addition of other good evaporation records, either past or future. It is believed, however, that the values given are better established than any hitherto presented, and that (until and unless they are replaced by other values still better established) they may be used with a fair degree of confidence as approximate measures of the correctness of other and differing assumed values.

Measured in this way, the values found by Mr. Post at San Diego (and shown in Column (3) of Table 89) do not appear to be justified at other places, either for full years, for separate months, or for any of the progressive periods of from 2 to 7 months included in Table 89, as his values of Column (3) vary from the (present) standards of

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Column (1) by from + 93 to — 10% for the separate months, by from + 68 to + 4.5% for the seven progressive periods, and by + 28.8% for the full year.

The only explanation offered by the writers, for the discrepancy between Mr. Post's values and the presumably better established general values of Column (1), is that given tentatively hereinbefore: that his values are "due to the influences of some unknown and unrealized local differences of climate and exposure, as between his land-pans and his floating-pans".

The foregoing comments on Mr. Post's values of (*b*) apply also in the main to Mr. Ledoux' value of Table 87.

As to Mr. Hawgood's opinion (that there are no stable relations between the evaporation depths from floating-pans and those from land-pans), the varying values of Column (1) of Table 89 at first glance seem to confirm that view, but a closer study seems to show that the variations of Column (1) are less than they may appear. For the separate months, the mean of the twelve monthly values is 82.5%, the maximum value is 95.9%, and the minimum value 60.8%, or a variation of from — 21.7 to + 13.4 per cent. For the seven progressive periods of from 2 to 7 months, the maximum value is 82.5% and the minimum 70.4%; or the variations from the yearly mean of 76.8% are from — 6.4 to + 7.7 per cent. These variations do not seem great enough to confirm Mr. Hawgood's view, even for parts of years.

The uniform value of 80% adopted by the writers compares with Column (1) as shown by Column (6). For the separate months, Column (5) varies from 92.1 to 141.4% of Column (1) and for the seven progressive periods from 97.1 to 113.7%; or the values of Column (5) vary from those of Column (1) from — 7.9 to + 41.4% for the separate months, from — 2.9 to + 13.7% for the seven progressive periods, and, for the full yearly cycle, is 4.2% greater. Hence the uniform value of 80%, assumed by the writers, is not (for the yearly cycle or even for the seven shorter periods) greatly in disagreement with the more carefully worked up values of Column (1).

Subsidiary Conclusion (c) of Table 2.—This conclusion (that the evaporation depth from a large reservoir is about 62% of that from a 3-ft. pan floating thereon) is that of Professor Bigelow, as interpreted by the writers.

Mr. Lee questions the foregoing statement* as not being a correct interpretation of Professor Bigelow's meaning, and dissents from the writers' statement that:

"* * * it has been recognized more or less generally that the evaporation losses from large reservoirs * * * are materially less than the evaporation depths measured in pans, even when the pans are floating on the reservoir".

* pp.1923–24.

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That others have interpreted Professor Bigelow's experiments in this respect in agreement with the writers' interpretation, and that such interpretation is a matter of public record, is shown by the following quotations*:

"Experiments made in 1909-10 by the U. S. Department of Agriculture gave the following figures for the annual evaporation at twenty places in the United States, the evaporating pan being at or very near the surface of the ground or water."

* * * * * * *

"Birmingham, Ala., 4 ft. diameter of pan, floating in reservoir; annual evaporation 51.74 inches."

* * * * * * *

"The evaporation from a pan 2 ft. in diameter is about 75%, that from a pan 4 ft. in diameter is about 50%, and that from a pan 6 ft. in diameter is about 30% greater than the evaporation from a large pond or lake. The above [tabular] figures may be roughly corrected by using these percentages; thus, at Birmingham, Ala., the true annual evaporation is 34.50 in."

From Table 13, it is seen that the foregoing percentages are equivalent to those of Table 2, and to the evaporation depth from a large reservoir, being 62% of that from a 3-ft. pan floating thereon. The Associate Editor of Section 13 of the "American Civil Engineers' Pocket Book" (from which the foregoing quotations are taken) is Mr. Louis A. Fischer, Chief of Division of Weights and Measures, U. S. Bureau of Standards.

Again, the relative evaporation depths from pans and from reservoirs is referred to (though rather indefinitely) by Philip Å Morley Parker,† M. Am. Soc. C. E., as follows:

"Modern studies show that the evaporation from the water surface of a large reservoir is considerably less than (usually about ⅔) that which is observed in ordinary evaporation pans."

(and)

"The later studies referred to on page 740, prove that the absolute magnitude of the evaporation from a free water surface has, until lately, been largely overestimated, and since no actual gaugings showing the loss assumed to exist have ever been recorded, I am inclined to believe that in Temperate Insular climates at any rate, the loss is inappreciable, or even non-existent."

Also, Mr. Hegly in his discussion‡ refers (as a well-known fact) to the smaller evaporation depths from reservoirs than from evaporation pans, though he does not mention whether he intends his statement to apply to land-pans, to floating-pans, or to both.

* "American Civil Engineers' Pocket Book", p. 1286 of 1st (1911), p. 1256 of 2d (1912), and p. 1256 of 3d (1916) editions.

† "The Control of Water", 1913, pp. 740, 207.

‡ p. 1991.

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As to the statements by Mr. Grunsky and Mr. Lee, that Professor Bigelow's Salton Sea experiments included no observations on floating-pans, and hence can furnish no data showing that the evaporation depth from reservoirs is materially less than that from pans floating thereon, it is shown in the preceding part of this closing discussion, on Subsidiary Conclusion (*a*), that Professor Bigelow's experiments did approximate floating-pan conditions. Also, he did not secure his value of C_2 for a large water surface by direct observation, but

*"By computing with the formula, assuming values of $C_2 = 0.021$, 0.023, 0.025, 0.027, 0.029, and using the observed temperatures of the Salton Sea surface water, it is now probable that the coefficient for a large water surface is $C_2 = 0.024$."

That value is 61.7% of his value, 0.039, for a 3-ft. pan.

This explanation applies also to Mr. Follansbee's discussion of (*c*).†

Aside from whether the writers have interpreted correctly in this respect Professor Bigelow's experiments, much more important questions are: whether any logical reasons can be given why the evaporation depth from pans floating on a reservoir should be materially greater than the evaporation depth from the reservoir itself; and whether such data as are available do or do not show the pan evaporation to be greater.

It seems to the writers that a sufficient reason is that given by Mr. Follansbee,† as follows:

"A contributing factor to the greater evaporation in floating-pans is the wetted strip around the inside of the pan, just above the normal water surface, caused by the wave action within the pan itself. This frequent wetting of the sides of the pan exposes an additional wetted area on a warm iron surface, and hence causes greater evaporation."

Some additional warm wetted surface must be caused by capillary action, and the waves within the pan and the area of the thin film of water on the warm iron surface must be increased greatly by the effect of the wave action in the reservoir and the resulting oscillatory tipping of the pan, even with small waves in the reservoir.

As to actual data tending to show whether or not the evaporation depth from a pan floating in a reservoir is greater than that from the reservoir itself, the only data known to the writers are those in the paper for Lake Conchos and Salton Sea, and some by Mr. Post and Mr. Whitney in the discussions.

Mr. Post's data relating to Murray Hill Reservoir‡ give the evaporation depth from that reservoir as 107% of that from a floating-pan; but the observations are for only a week, and the pan was floating on

* p. 1856.
† pp. 1927–28.
‡ p. 1905.

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La Mesa Reservoir, 5 miles distant. Due to the short duration, the distance of the pan from the Murray Hill Reservoir, and the possible differing climatic conditions at pan and at reservoir throughout the short period, it is not believed that these data can have any weight in determining this question.

Mr. Post's other applicable data are contained in Tables 58 and 59. Table 58 compares the computed actual monthly evaporation depths from Cuyamaca Reservoir with the evaporation depths from a pan 3 ft. square and 18 in. deep floating thereon. The observations cover a continuous period of 15 months, of which two disconnected months are excluded from the totals by Mr. Post for reasons not stated. The results of Table 58 may be summarized briefly as follows:

MONTHLY EVAPORATION DEPTHS FROM RESERVOIR, AS PERCENTAGES OF THOSE FROM FLOATING-PAN.

	Extremes.	Mean.
For the 15 months	35% and 118%	77%
For 13 months only (35% and 36% excluded)	45% and 118%	83%
Number of months less than 100%	11	9
Number of months more than 100%	4 15	4 13
Number of months less than 62%	5	3
Number of months more than 62%	10 15	10 13

Table 59 compares the actual monthly evaporation depths from Sweetwater Reservoir for 28 months (continuous) and from Upper Otay Reservoir for 7 months (not continuous), with the monthly evaporation depths from a pan, 3 ft. square and 18 in. deep, floating on La Mesa Reservoir, 7 miles from Sweetwater Reservoir and 12 miles from Upper Otay Reservoir. The results of Table 59 may be summarized briefly as follows, the percentages being for the "Mean" column of Table 59.

MONTHLY EVAPORATION DEPTHS FROM RESERVOIR, AS PERCENTAGES OF THOSE FROM FLOATING-PAN.

	Extremes.	Mean.
For the 28 months	3.2% and 127%	80 %
For 27 months only (excluding 3.2%)	31 % and 127%	82.5%
Number of months less than 100%	24	23
Number of months more than 100%	4 28	4 27
Number of months less than 62%	11	10
Number of months more than 62%	17 28	17 27

It is noticeable that the evaporation depths for the Upper Otay Reservoir, given by Mr. Whitney in Table 74, do not agree (for 2 of the 7 months) with those given by Mr. Post in Table 59; the depth in Table 74 being 8.0 in. for August, 1914, and 8.0 in. for August, 1915, and those by Mr. Post being 12.06 and 10.0 in., respectively. Accepting Mr. Whitney's values as the more probable and reasonable, and

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correcting the preceding table accordingly, the following values are obtained:

MONTHLY EVAPORATION DEPTHS FROM RESERVOIR, AS PERCENTAGES OF THOSE FROM FLOATING-PAN (CORRECTED).

	Extremes.	Mean.
For 27 months	31% and 121%	78%
Number of months less than 100%	24	
Number of months more than 100%	3	27
Number of months less than 62%	10	
Number of months more than 62%	17	27

It is apparent that the distances between pan and reservoirs (7 and 12 miles, in hilly country) make this average relation of 78% far from reliable. This is indicated, also, by the wide range in the monthly values, from 31 to 121 per cent.

The relations given by Mr. Whitney in Table 76, between the evaporation depths from the Upper Otay Reservoir and those from a 3-ft. square pan floating thereon, for 2½ months, have such wide variations (80% for October, 142% for November, 52% for December, and 87% as the average for the 2½ months), that they cannot be regarded with confidence as reliable measures of this relation.

As to Professor Bigelow's value of this relation at Salton Sea, interpreted by the writers as 62%, it is apparent that Mr. Louis A. Fischer, Chief of Division of Weights and Measures, U. S. Bureau of Standards, made an identical interpretation as early as 1911. The several rough checks given in Table 14 on the 62% value, though pointed out there as merely approximations, and with some uncertainties, still are based on values adopted after some thought by one physicist and two engineers, and are believed to have some corroborative value.

There remain for consideration only the Lake Conchos determinations of the value of (*c*), summarized in Table 50. As to the pan evaporations at Lake Conchos, they were made only by usual and ordinary methods, but, it is believed (as shown by Mr. Thorpe's discussion), with more care and attention to reliability than is at all usual in most engineering tests for evaporation; and, as representative of the true evaporation depths from the pans during the periods observed, it is believed that they may be accepted safely as at least as accurate as the majority of pan evaporation data accumulated up to the present time, and probably more so. The floating-pans were placed well out from shore, in exposed deep water apparently representative of the greater proportion of the lake; and, during the dry season (when the pan evaporations used were taken), the climatic conditions apparently were very uniform and stable over the area of the lake.

As to the measured evaporation depths from the lake surface itself, worked out in Table 45, the measurements on which these are based also were made with much care as to their reliability. Messrs. Duryea and Haehl.

The inflows to the lake as gauged at Pilar de Conchos and the outflows as gauged below the La Boquilla Dam, were from daily gauge heights in the low-flow season (of a stream not flashy and having drainage areas above the gauging stations of about 7 000 sq. miles), used in conjunction with carefully rated stream sections at the two gauging stations. The sections were re-rated afterward and found to be stable and unaltered, and the current meters (Gurley Price meters) were tested and found not to have changed their ratings. The outflows through the dam also were checked in other ways, as by pipe flow, and the seepage losses were measured carefully by weirs, etc.

The elevations of the lake surface (used in conjunction with its surface areas to estimate the increase or decrease in the volumes of stored water during the 7 months and the included 4-months periods) were taken from monthly tables of the daily gauge heights of lake elevation, which changed very slowly and evenly. The corresponding surface areas of the lake were from carefully prepared curves and tables of areas and volumes, and it is noticeable that, during the 4-month and 7-month periods, the changes of elevation were small, a net fall of only 0.04 m. (1.58 in.) in the 7-month period and a net rise of only 0.32 m. (12.60 in.) in the included 4-month period.

As to the elimination in Table 45 of the rain falling directly on the lake surface, and the substantial uniformity of the rainfalls at the opposite ends of the lake, such uniformity is shown by the comparisons in Table 90.

At both La Boquilla and Pilar de Conchos only a small part of the total yearly rainfall occurs during the 8 dry months, October to May, and about the same proportions at the two places. These facts are shown by Table 91.

Table 91 shows that, during the 12 months, June, 1913, to May, 1914, the rainfall at Pilar de Conchos was only about 10% greater than that at La Boquilla; and, during the 8 months, October, 1913, to May, 1914 (about the period for which the evaporation depth from the lake itself was estimated), only about 16% greater and 2.2 in. more. These close agreements and the small differences in the monthly rainfalls of Table 90 at the two places show the small probability of any material error arising in the estimation of the average rainfall on the lake as the mean of the rainfalls at La Boquilla and at Pilar de Conchos.

Another important element in Table 45 is the assumption that there were no accessions to the lake storage during the 7 months, October, 1913, to April, 1914, except the gauged inflow of the main river past Pilar de Conchos and the rain falling directly on the lake surface. That assumption seems justified, for several reasons.

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TABLE 90.—COMPARATIVE MONTHLY RAINFALLS, IN INCHES, AT LA BOQUILLA AND AT PILAR DE CONCHOS, 25 MILES APART.

(See Figs. 1 and 3.)

Month.	YEARS.							
	1912.		1913.		1914.		1915	
	La Boquilla.	Pilar de Conchos.	La Boquilla.	Pilar de Conchos.	La Boquilla.	Pilar de Conchos.	La Boquilla.	Pilar de Conchos.
January...	*0.28	Trace.	0.35	0.20	0.00	Trace.	0.51	1.04
February..	0.00	"	0.46	0.69	0.06	0.08	Trace.	0.06
March.....	0.00	"	Trace.	0.00	1.10	0.31	0.24	0.37
April.....	0.00	0.43	0.37	0.04	0.00	Trace.	0.04	Trace.
May.....	1.14	Trace	0.02	Trace.	1.26	1.02	0.00	Trace.
June.....	0.51	1.59	3.37	3.21	4.70	8.54	Trace.	0.41
July.....	2.05	1.78	2.59	1.73	3.15	3.60	4.36	4.42
August....	2.30	3.61	1.75	6.08	4.82	3.17	9.56	6.77
September.	2.53	4.08	4.21	3.09	2.42	4.92	2.72	3.99
October...	0.85	0.98	0.18	1.22	2.52	1.08	1.10	0.00
November.	0.30	0.49	1.61	1.10	2.22	1.77	0.00	0.00
December.	0.26	0.28	0.12	0.28	1.14	1.14	Trace.	0.08
Year.....	10.22	13.24	15.03	17.64	23.29	25.63	18.53	17.14

* The rainfalls were measured and recorded in millimeters and half-millimeters, and reduced for this table at the rate of 1 in. = 25.4 mm.

TABLE 91.—COMPARATIVE DRY-SEASON RAINFALLS, IN INCHES (OCTOBER TO MAY, INCLUSIVE), AT LA BOQUILLA AND AT PILAR DE CONCHOS.

Rainfall year.	LA BOQUILLA.			PILAR DE CONCHOS.		
	12 months.	June to September.	October to May.	12 months	June to September.	October to May.
1912-13.....	10.00	7.39	2.61 = 26%	13.74	11.06	2.68 = 20%
1913-14.....	16.25	11.92	4.33 = 27%	18.12	14.11	4.01 = 22%
1914-15.....	21.76	15.09	6.67 = 31%	25.69	20.23	5.46 = 21%
Mean.....	16.00	11.47	4.53 = 28%	19.18	15.13	4.05 = 21%

Rainfall year.	12 months. La Boquilla ÷ Pilar = Percentage.	8 months. October-May. La Boquilla ÷ Pilar = Percentage.
1912-13.....	10.00 ÷ 13.74 = 73%	7.39 ÷ 11.06 = 67%
1913-14.....	16.25 ÷ 18.12 = 90%	11.92 ÷ 14.11 = 84%
1914-15.....	21.76 ÷ 25.69 = 85%	15.09 ÷ 20.23 = 75%
Mean.....	16.00 ÷ 19.18 = 83%	11.47 ÷ 15.13 = 77%

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First, the region is a semi-arid one, with no small running streams throughout the dry season, when water is flowing only in large rivers like the Rio Conchos. This condition is a usual and general one, and such arroyos as lead into the lake between Pilar de Conchos and La Boquilla are dry, except during the heavy rains of the rainy season. This condition was observed by Mr. Thorpe and his assistants at frequent intervals, and even by one of the writers (on a trip from La Boquilla to Pilar de Conchos and return, in March, 1914). Also, any material inflows from those arroyos carry with them considerable silt, and thus increase quickly and noticeably the turbidity of the lake.

Second, the drainage areas of the arroyos leading into the lake from the north and south are narrow and small, and can easily absorb all such small and infrequent rainfalls as occur during the dry season. This is evident from the comparative data of rainfall and run-off as follows:

From Tables 97 and 98, and more detailed data, the total stream flow of the Rio Conchos during the whole year, 1913, was only a very small proportion of the rainfall on its drainage areas:

From drainage area above its mouth only about 4.5 per cent.
" " " " La Boquilla " " 6.8 " "

The greater part of these yearly run-offs occurred during the rainy and flood season, and nearly all of that measured at La Boquilla came from the (about) 7 000 sq. miles of drainage area up stream from the lake, and very little of it from the few square miles of tributary area abutting on the lake; and, during the dry season, the percentage of run-off from those abutting areas must have been practically nothing.

During the 12 months, May, 1913, to April, 1914, the total measured run-off at La Boquilla (including the floods of May to September, 1913), was equivalent to an average depth from the (about) 7 000 sq. miles of drainage area of only about 1.11 in.; of which only about 0.12 in. (about 11%) ran off during the 7 months, October to April. It is believed that all of this 0.12 in. was furnished from the large area up stream from Pilar de Conchos, and practically none of it from the two small narrow drainage areas at the north and south sides of the lake, between Pilar de Conchos and La Boquilla Dam.

From Table 6, the normal or mean rainfall at the lake during the 7 months, October to April, amounts to only about 2.75 in., or about 23% of the mean yearly rainfall. From Tables 45, 90, and 91, the rainfall at the lake during the 7 months, October, 1913, to April, 1914, was only about 3.0 in. From Table 90 and daily rainfall records, the infrequent rainfalls of the dry season are widely distributed over the 7 months, with ample opportunity for the ground to dry out between the rains, and hence with very little chance for those rains to contribute to the run-off or stream flow.

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As the result of 7 years of very careful rain and stream gaugings, by the writers, of a California stream having rainfall and seasonal conditions quite similar to those of the Rio Conchos (the Coyote River, of about 200 sq. miles drainage area), it was found that there were no stream flows contributed by the scattered autumn rainfalls of a new rainy season until after the first 2 to 10 in. (usually from 5 to 8 in.) of rain had fallen. This condition was confirmed by similar gaugings, of 1 to 4 years each, on six other streams in the same general region, the rainfalls, before stream flows resulted, being there 1.3 to 13 in., usually from 5 to 10 in. The average regional yearly rainfalls on the Coyote drainage area varied from about 19.0 to 35.4 in. during the 7 years, and on the other six streams from about 18.3 to 50.0 in. The drainage areas of all the seven streams are in general rough and mountainous, and the year's rainfall nearly all occurs in a rainy season (about October to April) of about 6 months.

From what has been said it is apparent that:

The stream flows at La Boquilla for the calendar year, 1913, aggregated only about 6.8% of the tributary rainfall, most of this stream flow occurring in the rainy or flood season of June to September, and nearly all of it coming from the large tributary area (about 7 000 sq. miles) above Pilar de Conchos;

During the 12 months, May, 1913, to April, 1914, the total stream flow or run-off at La Boquilla was equivalent to only about 1.11 in. depth on the tributary drainage area, of which only about 11% (about $\frac{1}{8}$ in.) occurred during the 7 months, October to April;

In those 7 months only about 3 in. of rainfall occurred at Lake Conchos, in scattered rains;

Under similar conditions, on similar streams in California, usually more than 5 in. of scattered rains fell in the autumn before any of it began to appear as stream flow; and

From occasional visual observations along the lake and constant observations at the dam, there is believed to have been no material inflow into Lake Conchos between Pilar de Conchos and the La Boquilla Dam during the 7 months, October, 1913, to April, 1914.

About the only other elements which enter into the Lake Conchos check of the 62% value of Subsidiary Conclusion (*c*), are the reduction by 20% of the observed evaporation depths from the land-pans to bring them to assumed corresponding depths from floating-pans, and the expansions into full years of the (about) 5 months pan observations and of the 7 months lake observations. As to the 80% relation, that has been discussed already under (*b*), and it is believed to be fairly well established that the evaporation depths from floating-pans are in gen-

eral considerably less than those from near-by land-pans of the same size, and probably about 80% thereof. As to the expansions from part year to full year, they were each made by two methods, (*a*) and (*b*) (B_a and B_b for the pans, and C_a and C_b for the lake), the two methods checking each other closely for yearly evaporation depths; and, as the periods for pan and lake observations were largely coincident (January to June for pans and October to April for lake), and as the same methods of expansion were used for both, the correctness of their comparative yearly values (or the value used to check Subsidiary Conclusion (*c*)) should to a large degree be unaffected by the absolute correctness of the methods of expansion used and of the resulting yearly values. Messrs. Duryea and Haehl.

In view of all the additional explanations of Subsidiary Conclusion (*c*) which have been given, at the very least, it is believed to be a reasonable probability that the evaporation depths from large reservoirs are much smaller than those from evaporation pans floating thereon, and for 3-ft. pans probably only about 62% as great. It is acknowledged, however, that this and some others of the writers' conclusions are shown by some of the discussions to be open to more or less question, which can be settled conclusively only by more complete, exact, and reliable future data. In the meantime, however, it is felt that Subsidiary Conclusion (*c*) may be given at least tentative acceptance as expressive of the best data as yet available.

The remaining subsidiary conclusions of the paper are that the evaporation depths at different places throughout the Great Plateau increase almost directly with (*d*), increases in the mean temperature, and (*e*), increases in the elevation above sea-level, as stated in Table 2 and shown by Fig. 4. This discussion of (*d*) and (*e*) will be more or less a combined one, and also will be briefer than the foregoing part of this closure.

It is apparent from a study of the paper that (*d*) and (*e*) are based entirely on data of pan evaporations made at six stations, and not at all on Piche evaporimeter records and evaporation depths calculated therefrom. The Piche records presented in the paper were considered for use in conjunction with the pan data, but were rejected as unreliable for such uses. Many interesting comments on Piche records are given by Messrs. Lee, Follansbee, Meyer, Comstock, Grunsky, and Hegly, all of whom seem to agree on the unreliability of Piche records as measures of absolute evaporation depths; however, no actual use was made of them in the paper.

In considering (*d*) and (*e*), it should be remembered that they were stated repeatedly by the writers as applicable only to the evaporation relations between places on the Great Plateau, or between places where the climatic conditions other than mean temperature are quite similar. Apparently, the writers did not emphasize this point to the satisfaction

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of Mr. E. F. Chandler,* though they intended to emphasize it and thought they had done so.

Conclusion (*d*) seems to be agreed with in a general way (at least qualitatively, even if not quantitatively) by several of those who discuss it. Mr. Chandler shows by Fig. 16 that, for a period of 10 years in North Dakota, with observed values for 71 months, the observed evaporations are found

"* * * to be satisfied fairly well from temperatures 35 to 75° Fahr. by the 'straight-line' curve representing the equation,

$$E = 0.135\ T - 4.00,$$

in which E is the evaporation, in inches, during the month, and T the mean temperature of the month, in degrees, Fahrenheit."

It also is stated by him that, if the values of Fig. 4 and Fig. 13 are applied to the mean monthly temperatures of the Red River Valley of North Dakota, the corresponding equation is

$$E = 0.20\ T - 10.00.$$

As was to be expected, this differs widely from his equation of North Dakota evaporations and mean temperatures.

Mr. Post shows, by Figs. 17 and 18, that for the climatic conditions near San Diego, Cal., the relations between evaporation, mean temperature, and elevation above sea level may be represented fairly well (neglecting all the other elements of the climate) by straight lines or by lines of light curvature approximating them.

Mr. Lee says†:

"The authors' analysis of this subject [(*d*)] is very interesting and suggestive, and the results are of value within the general region to which they can be applied and when evaporation records are not available."

As to (*e*), he says:

"The writer would make much the same comment * * * as he has on [(*d*)]."

Mr. Robert Follansbee‡ seems to accept the writers' Conclusion (*d*), in so far as it relates to the southern part of the Rio Grande drainage, but to find it not applicable to some other regions which he has investigated, in this respect. He says:

"The writer was especially interested in the simple relations between evaporations and temperature for the records in the southern part of the Rio Grande drainage. To determine whether a similar relation

* pp. 1903–04, 1906.
† p. 1924.
‡ pp. 1928–30.

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exists for the upper part of the Rio Grande drainage at an elevation of 7 200 ft., the records near Santa Fé were investigated, [as] compiled by the U. S. Geological Survey.

* * * * * * *

"* * * Fig. 21 shows that the points do not lie on or near a single straight line, as would be the case if the simple relation between monthly evaporation and temperature obtained.

* * * * * * *

"* * * From these records, it is evident that other factors than temperature—such as wind velocity and relative humidity—influence evaporation to such an extent in the upper part of the Rio Grande drainage that it is impossible to determine evaporation [there] with any degree of accuracy from temperature records alone.

"An investigation of evaporation records in the southwestern part of Oklahoma was made in order to determine whether a simple relation obtained between temperature and evaporation in that section of the country.

* * * * * * *

"The plotting of the monthly evaporation and mean air temperature, Fig. 22, shows that the relation between the two at this point is complicated to a great extent by the other evaporation factors, making it impossible to determine evaporation [there] from air temperatures alone."

Mr. Grunsky states that some years ago he himself pointed out:

"* * * the preponderating influence of temperature on evaporation * * * and * * * the use of mean monthly temperatures as a guide in approximating the evaporation from a water surface such as that at Salton Sea was illustrated. This article* was prepared by the writer in order to show that, throughout regions in which climatic conditions are similar, it may be assumed that errors due to more or less wind movement in one locality when compared with another, and those due to small differences in relative humidity and other similar causes, may be neglected, and, [that] for all preliminary engineering purposes, evaporation from an open water surface may be determined from mean monthly temperature.

"It will be apparent, however, that this can only be done after the relation between the mean monthly temperature and the rate of evaporation at one or more selected points during the calendar months has been determined experimentally."

Mr. Grunsky thus agrees that the evaporation does increase as some function of the mean temperature (as is almost self-evident), and that the influence of mean temperature is "preponderant"; but he does not agree with the writers' Conclusion (*d*) as to a simple "straight-line" relation.

As to the straight-line relation: the writers must admit that it is somewhat forced, and is only approximately true; and that, in view of the few data used by them as a basis for it, and of the uncer-

* *Engineering News*, Vol. 60, August 13th, 1908, p. 163.

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tainties of some of these, which have been pointed out, the straight-line relation does not seem well proven.

However, Mr. Grunsky's Figs. 28, 29, and 30 show that, for mean monthly temperatures greater than 50 to 60° Fahr., the curves of temperature evaporation relation are light ones and not great departures from straight lines, even for the long, careful, and widely separated evaporation observations at Kingsburg, Chestnut Hill, Lee Bridge, and Owens River; and Table 42 shows that the evaporations observed at Lake Conchos differ but little from those estimated by assumed straight-line temperature-elevation-evaporation relations. However, this close agreement, of course is partly dependent on some of the assumptions and data which have been criticized adversely by some of those discussing the paper.

In connection with Conclusion (*d*), it is of interest to note that in Table 71 Mr. Grunsky has estimated the normal yearly evaporation depth from Salton Sea by using the monthly mean temperatures there in conjunction with the Kingsburg evaporation-temperature curve (presumably Fig. 28), and neglecting all other elements of the two climates; and gets as a result a probably normal yearly evaporation depth from Salton Sea of 6.24 ft. (or a little less), or about 74.8 in. From Table 14, various estimates of the yearly evaporation from Salton Sea are from 67 to 73.7 in., or in fairly close agreement with Mr. Grunsky's estimated value of 74.8 in.

In Table 71 there are no normal mean monthly temperatures at Salton Sea below 50° Fahr., and only 3 months (December, January, and February) appreciably below 60°; and the light curve of Fig. 28 is approximately a straight line above 60°, and only a little farther from it above 50 degrees. Hence it appears probable that, as between Kingsburg and Salton Sea, the yearly evaporation may be estimated with a fair degree of approximation by a "straight-line relation" of evaporation to mean temperature, neglecting all the other elements of the two climates. Salton Sea is about 300 miles from Kingsburg, and is about 200 ft. below sea level, or about 500 ft. lower than Kingsburg. The two places are separated by much mountainous country, and presumably have considerable differences of climate.

This close check may be merely an accidental coincidence; but it appears to the writers to have some weight (even though indefinite), as indicating the comparatively small influence on evaporation of most elements of climate other than mean temperature, and the approximate "straight-line relation" between evaporation depth and mean temperature.

Others who discuss Subsidiary Conclusion (*d*) are Messrs. Comstock, Horton, and Ledoux. Mr. Comstock's study* of the mean departures of the observed monthly evaporations at El Paso, Albu-

* p. 1959.

querque, Carlsbad, and Elephant Butte from their respective curves is instructive, and weakens those data somewhat as a valid basis for Conclusion (*d*). However, though the mean departures of 12 to 15% are in a sense large, as related to values in climatology they are not unduly large. For instance, in Tables 28 and 29 the mean departure of the 11 yearly values of pan evaporation in the Santa Clara Valley is 11%, of the 6 yearly values at Lake Tahoe 8%, and of the 11 Emdrup values 6%, with maximum departures of 25, 16, and 14%, respectively; and, for the 16 years' evaporation records at Boston, the maximum departure was 13 per cent. The three evaporation records just cited are believed to be as reliable, at the very least, as is usual for data of this class; and the mean departures of 12 to 15% pointed out by Mr. Comstock are but small compared with the greater departures usual in records of yearly rainfall and stream flow. The 12 to 15% mean departures must be admitted to weaken Conclusion (*d*) as dependent on those data; still, the data are felt to give some weight to that conclusion, though (*d*) is admitted to be more roughly approximate than was realized. Messrs. Duryea and Haehl.

Discussing Subsidiary Conclusion (*d*), Mr. Robert E. Horton says*:

"It has long been known that measured evaporation losses from water surfaces could be represented graphically in a fairly satisfactory manner in terms of temperature alone as the independent variable.

* * * * * * *

"For the purpose of determining the relation of evaporation to temperature, considering the latter as the only independent variable, the writer has tabulated the experimental data for Albuquerque, N. Mex., from the records of the Hadley Climatological Laboratory, as appearing on Plate LXII, and has computed therefrom the correlation coefficient. * * * The computations are given in Table 78, and indicate a correlation coefficient, 0.92 in this instance. The probable error of a single observation by the usual formula is 0.0189. If there is perfect correlation between two quantities, the correlation coefficient should be unity. If there is no correlation, it would be zero. If the correlation coefficient is five to six times as great as the probable error and is also greater than one-half or two-thirds in absolute value, it indicates a considerable degree of correlation. This calculation appears to indicate that evaporation in the long run may be treated as though it were a function of temperature alone at a given locality. It is true, however, that in any individual period, as for a given month or year, the evaporation is very profoundly modified by other factors, such as variation in the wind velocity and vapor pressure in the air."

Mr. Ledoux says†:

"In 1888 and 1889 the writer made weather observations, which included the measurement of evaporation on a water surface containing about 25 acres, in the Alleghany Mountains.

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* pp. 1994–96.
† pp. 1998–2000.

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"Up to that time, the writer was under the impression that the evaporation from water surfaces was more directly related to relative humidity than to any other factor, but one month's observations absolutely dispelled this belief.

"The final conclusions were that evaporation depends almost entirely on the temperature and the wind, and has very little relation to humidity, even where the relative humidity varies between 70 and 95%; * * *

* * * * * * *

"The writer is under the impression that the evaporation from a water surface is affected materially by humidity where the relative humidity is less than 70 per cent. * * *

" * * * that land gauges give the same result as floating gauges, provided the temperature of the water in the evaporating vessel is kept the same as that of the water in the reservoir; that a gauge 6 in. in diameter will show as much evaporation as one 18 in. in diameter; that it is important that the evaporating vessel be kept full, so as to get a complete water exposure; * * *

"The evaporating vessel was kept as full as possible, say, within ½ in. of the top."

The conclusions of Mr. Ledoux are of great interest, and seem to show the preponderating influence on evaporation of temperature and the small influence of humidity, and thus to strengthen (*d*) somewhat. Some of his conclusions are opposed to the rather general beliefs of other engineers, however, and, in view of this, it is regretted that he did not present the data on which they were based.

His Fig. 31 is of much interest as showing the very great increase in the evaporation depth from increasing the temperature of the water in the evaporating vessel while the air temperature is constant. However, this, of course, is an unnatural condition, as in Nature the temperature of exposed water bodies must follow that of the air, up and down, more or less closely, though with considerable "drag", and with less widely separated extremes of temperature.

It is noticeable that the curve of Fig. 31 is of rather light curvature between temperatures of 80 and 110°, and could be represented there fairly well by a single straight line.

His conclusion that the evaporation depth from small vessels is no greater than that from larger ones is not only opposed to the results of Professor Bigelow's Salton Sea observations, but also is opposed to the natural and very probable influence of the rim conditions which have been referred to hereinbefore, since the rim influence necessarily must increase the evaporation depth as the size of the evaporating vessel decreases. Keeping the vessels filled almost to their rims, as he recommends, of course will lessen the influence of the rim conditions on the evaporation depth; but, under natural exposed conditions (and especially with pans floating in reservoirs), it is hard to see how pans can be kept so completely filled without much risk of frequently

vitiating the record because of the water slopping in and out of the pan from wind and from wave action. Messrs. Duryea and Haehl.

All the evaporation tests given by Mr. Ledoux appear to have been made in vessels so small as to be outside the class of evaporation pans used for practically all recorded observations, which makes his results of doubtful applicability for combinations and comparisons with evaporation records secured from the usual larger pans.

As to Subsidiary Conclusion (*e*), Mr. Grunsky says*:

"The writer believes that the rate of evaporation in relation to mean monthly temperature increases with elevation in some measure, as shown by the authors on Table 17. It is reasonable to expect this, not only because the temperature at which water boils falls with increasing altitude, but, also, because the temperature range, particularly the diurnal range, ordinarily increases with altitude. According to the evaporation law, the mean monthly rate of evaporation will be greatest at that place which shows the greatest temperature fluctuations, because each degree of temperature above the mean produces more evaporation than each degree below the mean.

* * * * * * *

"Although it may be admitted that, for the same mean monthly temperature, the evaporation is greater at high than at low altitudes, the law of this increase is not to be regarded as proven by the data presented in the paper, and Conclusion (*e*) * * * should not be accepted without further verification."

Mr. Grunsky's suggestion of the greater evaporation depths caused by temperatures above than by those below the mean, is illustrated very clearly by Fig. 31. His statement that the writers' Subsidiary Conclusion (*e*) is not to be regarded as proven by the data presented by them in the paper seems to be warranted, especially in view of the discussion of (*e*) by Mr. Meyer, who points out that some of the differences of evaporation between the Texas and New Mexico stations, assumed by the writers to be due to differences of elevation, as well may be ascribed to differences in their probable humidities.†

Referring to Mr. Meyer's observation that from Fig. 4 apparently there would be no evaporations at places on the Great Plateau with

Altitudes about	3 000-4 000 ft.	and mean temperatures	below	30°	Fahr.		
"	" 2 000-2 500 "	"	"	"	"	40	"
"	below 1 000 ft.	"	"	"	"	45	"

This certainly is a serious defect in the diagrams, if their curves are assumed as producible to such low mean temperatures at such low altitudes. However, the diagrams as published are restricted to lower limits of 40° and 4 800 ft., and 45° and 4 000 ft., and to extremes of 600

* pp. 1981-82, 1980.

† pp. 1932-34, 1943-1946.

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and 5 000 ft. altitude, 40° and 90° mean temperature, and 2 and 12 in. monthly evaporation (in 3-ft. floating-pans); and no surmises are made in Fig. 4 as to evaporation depths beyond such limits.

Incidentally, it is apparent from Table 85 that there are almost no mean yearly temperatures (irrespective of altitude) lower than about 50° Fahr. at any of the nineteen places there tabulated, and only two places that low (50° and 48.9°); and even the January mean temperatures are as low as 40° Fahr. at only six of the nineteen places. Of course, there must be some evaporation at 40° and even at still lower temperatures; but it must be small and of little importance as affecting the total yearly quantity.

However, Mr. Meyer's discussion does throw some doubt on the influence of altitude in itself (apart from humidity, etc.) on evaporation depth, and the writers cannot but assent to his conclusion, that*:

"* * * records of relative humidity and wind velocity, combined with records of temperature, afford a far more accurate and reliable basis for estimating evaporation from reservoirs than Fig. 4."

They still believe, however, that altitude should be included as one of the elements influencing evaporation.

Mr. Comstock, though dissenting from the writers' Subsidiary Conclusion (*e*), seems to assent to their claim that evaporation depth increases in some manner with increases of elevation above sea level. In discussing (*e*), he says†:

"* * * Professor Bigelow's failure to find a barometric term [in his equation for evaporation depth] * * * is at variance with experience and experiment.

"In 1789, H. B. de Saussure made comparative observations * * * from which he concluded that, 'other things being equal, a lowering of the pressure of the air by approximately a third makes the quantity of evaporation more than twice as great'.

"Numerous laboratory experiments have shown beyond question that the air opposes a mechanical obstacle to the escape of liquid particles in the form of vapor, and that this obstacle is greater at high than at low pressures.‡ It has been usual to introduce the barometric pressure in such a way as to make the rate of evaporation inversely proportional to the pressure. This is arbitrary, and, although perhaps sufficient to cover all natural conditions, is certainly not correct, as it would result in an infinite rate of evaporation in a vacuum, no matter what the temperature. However, if it is even approximately correct, the relation sought by the authors between evaporation and altitude above sea level must be very complex, because it contains implicitly the relation between altitude and barometric pressure.

"The authors have not established any explicit relation between altitude (or barometric pressure) and evaporation, but have put

* p. 1945.

† p. 1965.

‡ Hence greater at low than at high altitudes.

forward a double relation involving altitude, monthly mean temperature, and evaporation. Since, for reasons already given, the temperature-evaporation relation must be rejected, the altitude-temperature-evaporation relation falls with it." Messrs. Duryea and Haehl.

Mr. Comstock's reference to Conclusion (*e*), as being "clouded by the intimate admixture of the assumed evaporation-temperature relation * * * by a double relation involving altitude, monthly mean temperature, and evaporation", hardly seems justified. Irrespective of the validity of (*e*), it is apparent from an inspection of Diagram (*b*), Fig. 4, that especial care was taken to keep the supposed independent influences of elevation and of temperature separated and simplified, by showing the separate effects of altitude at many different temperatures. After considerable thought, no better way could be devised to do this than by Diagram (*b*) of Fig. 4.

In final comment on Subsidiary Conclusions (*d*) and (*e*), it is felt by the writers that they still may be regarded as at least first approximate expressions of probable general relations—even though, in the discussions, some valid and important objections to (*d*) and (*e*), and to their underlying data and methods, have been raised. It is admitted that, as yet, (*d*) and (*e*) are open to more or less question, and that more extended and more accurate data are desirable; but, notwithstanding the admitted faults and doubts, it is felt that the amenability of evaporation-temperature-elevation data to expression in such diagrams as Figs. 4 and 13 (Texas and New Mexico data), Figs. 17 and 18 (San Diego data), and Fig. 16 (North Dakota data, one elevation only), in conjunction with some of the various opinions quoted, make Subsidiary Conclusions (*d*) and (*e*) still worthy of further consideration and investigation.

There remain for consideration various minor points mentioned in the discussions; and the discussions relating to the Principal Conclusion of the paper, the probable yearly evaporation depth from Lake Conchos.

The data of evaporation near Morelia, Michoacán, Mexico, furnished by Mr. Mathewson,* are of much interest as a check on the applicability of Fig. 4. From and through Mr. Mathewson, the following additional data were furnished to the writers:

"The rain gauge at Morelia had a diameter of 0.226 m. (8.90 in.). I am under the impression that the evaporimeter (evaporating vessel) was of that diameter, but am not sure. Only one evaporimeter was used, a land one located at Morelia with its bottom and sides protected by earth and sod; and the evaporation record secured from it was from May, 1908, to May, 1912, a period of 4 years.

"We observed the farm reservoir about a month and a half, at a time of year when evaporation was high, near the end of the dry season of 1911, I think in April and May. During that period the

* pp. 1902–03.

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evaporation from the farm reservoir was only 52% of the coincident evaporation from the land evaporimeter at Morelia. However, the 52% obtained was applied to the evaporimeter data of a drier year, 1909, the year of highest observed evaporation. We used no other data in arriving at a value for evaporation depth from reservoirs".

TABLE 92.—OBSERVED MEAN MONTHLY TEMPERATURES OUTSIDE OF BOTELLO POWER-HOUSE (ALTITUDE ABOUT 6 560 FT.) NEAR MORELIA, MICHOACÁN, MEXICO, FOR 1915; ALSO, CORRESPONDING MONTHLY EVAPORATION DEPTHS FROM DIAGRAM (*b*), FIG. 4.

Month of year 1915.	Observed mean temperature, in degrees, Fahrenheit.	Corresponding monthly evaporation from 3-ft. floating-pans (as read from Diagram (*b*), (Fig. 4), in inches.
January	* 55.2	7.37
February	55.8	7.50
March	60.0	8.38
April	65.1	9.48
May	71.4	10.43
June	71.0	10.35
July	67.8	9.80
August	66.5	9.62
September	64.5	9.19
October	59.0	8.17
November	57.8	7.92
December	54.8	7.28
Sum	748.9	105.49 in. for year.
Mean	62.4	8.80 in. per month.

* January, 1916 = 57.4° Fahr.

Observed mean monthly temperatures outside of the Botello power-house ("about 4 miles from the farm reservoir observed and a long day's horse-back ride from Morelia") were furnished through the kindness of Mr. Mathewson (see Table 92). He states that as between Morelia and the power-house "there is little if any difference in altitude (about 2 000 m. or 6 560 ft. above sea level) or in climate, and other conditions are similar". To Table 92, for use hereinafter, are added the monthly evaporation depths corresponding to its monthly mean temperatures, as read off from the original larger scale drawing of Diagram (*b*), Fig. 4, extended by straight lines to altitude 6 560 ft.

The conclusion arrived at by the engineers of the Botello power-house was that the evaporation depth from their reservoirs would have been 1.72 m. = 5.64 ft. = 67.7 in. for 1909, the year of greatest evaporation among the 4 years observed by the evaporimeter at Morelia.

Morelia (not far from the proposed reservoirs) is near the southern end of the Great Plateau, about 600 miles in an air line from La Boquilla.

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From Table 92 and Fig. 4, the probable evaporation depths at the Botello power-house in 1915 should have been (from 3-ft. floating-pans) about 105.5 in. for the year, or an average of about 8.80 in. per month. As a check, from Fig. 4 the evaporation depth at altitude 6 560 ft., for a mean temperature of 62.4° Fahr. (the altitude and mean temperature at the Botello power-house), is 8.88 in. per month, or about 106.6 in. per year.

The value of, say, 106 in. per year, multiplied by the 0.62 adopted by the writers as Subsidiary Conclusion (*c*), gives a corresponding evaporation depth from reservoirs near Morelia, Mexico, for 1915, as 65.7 in., as compared with the 67.7 in. adopted there by the engineers from local observations of evaporation, and for 1909, stated to have been the year of greatest evaporation in the 4 years, 1908-11, which were observed (and perhaps greater in evaporation than 1915).

Of course, the apparently close check (of 65.7 in. for 1915 with the 67.7 in. adopted for 1909) may be merely accidental, but, at the least, it does not weaken the writers' statement in the paper, that, in general, fair approximations to evaporation depths at places on the Great Plateau may be read off directly from Fig. 4, without other data than mean temperatures and altitudes.

Mr. Mathewson seems to agree with the writers' belief as to "the almost complete absence of meteorological data in Chihuahua". Mr. Horton calls attention to many evaporation data there and elsewhere in Mexico,* and is to be congratulated on his knowledge of and possession of such little known data. Their existence was unknown to some well-known engineers who had done important hydraulic work in Mexico, and of whom the writers inquired, and is believed to have been unknown to most American engineers.

However, since the evaporation pans used for these Mexican observations were only 8.86 in. in diameter and only 4 in. deep, and were on the roofs of houses and in corridors, it is not felt that these data are of much value to engineers as showing absolute depths of evaporation, or in any except comparative ways, such as those mentioned by Mr. Mathewson and by Mr. Ledoux.† It is felt that observations conducted with such care, in pans so small and so far outside the range of sizes used almost universally for observations of evaporation, show much wasted effort, though, of course, they are of some value for comparative purposes, when data from larger pans, of usual size, are not in existence.

Mr. Mathewson's determination of the evaporation depth from the farm reservoir as 52% of that from the small land evaporimeter is of little interest, aside from the local use made by him. If Subsidiary Conclusion (*b*) should hold true for such small vessels, which is very

* pp. 1996–97.

† pp. 1998–2000.

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doubtful, then the evaporation depth from the farm reservoir was (52 ÷ 0.80) = 65% of that from an 8.9-in. evaporimeter floating on it. However, no confidence is felt in the applicability of (*b*) to such small vessels, so far outside the range of sizes usually observed.

Mr. Chandler's data and discussion of observed pan evaporations in North Dakota*; Mr. Follansbee's evaporation data in Oklahoma†; Mr. L. R. Jorgensen's conclusion as to the evaporation depth from Silver Lake, Mono County, California, as 54 in. per year‡; and Mr. Hegly's data of evaporation in Northern Africa and in France§, are all of much interest and value, as easily accessible data of evaporation are rather meager.

Mr. Meyer presents interesting and valuable matter relating to Piche evaporimeter observations, and to Russell's formula and evaporation table, etc.‖ Figs. 23 to 25 compare the Russell and Duryea-Haehl curves of monthly evaporation and mean temperature, and show them to disagree more or less with each other, as mentioned by him; but still show a rough general agreement in form, both being usually of light curvature, approximately parallel to each other, and not greatly removed from each other in values. Also, some of these curves are for places not on the Great Plateau.

Plate LXVIII, showing contours of elevations and values of relative humidities for many places on the Great Plateau, is of much value, as is his pointing out the excessive humidity of Pike's Peak, compared with that of Fort Grant, about 9 000 ft. lower in altitude, etc.

Mr. Comstock's discussion of the Bigelow formula and of the probable effect of winds on the vapor blanket¶ is of much interest, especially his quotations from the writings of Professors S. P. Langley and Cleveland Abbe. Mr. Comstock is in error, however, in assuming that the writers made any use of, or gave any consideration to, the Bigelow formula, other than by accepting his relative values of the coefficient, C_2 (apparently the principal and controlling factor of the formula, for the uses made of it by the writers); and those values are believed to be based on local experimental observations of the evaporation depths, rather than on theory.

Mr. Horton states**:

"In this paper the specific result [the yearly evaporation depth from Lake Conchos] seems first to have been arrived at by guesswork. The authors, by repeatedly calling attention to this guess, as if it in reality was of some value as corroborating their results, have,

* pp. 1903–06.
† pp. 1929–30.
‡ pp. 1931–32.
§ pp. 1991–94.
‖ pp. 1932–46.
¶ pp. 1948–54, 1961–63.
** p. 1997.

unfortunately, and no doubt unintentionally, created the appearance that their investigation was for the purpose of corroborating the original guess. * * * it seems preferable to avoid if possible pre-influencing the mind by having any fixed figure set up in advance." Messrs. Duryea and Haehl.

The writers regret that Mr. Horton should have gained that impression, for, as a matter of fact, they had no preconceived ideas as to the evaporation depth from Lake Conchos, and no choice between the value of 55 in. of Mr. Freeman and that of 83 in. of Mr. Stearns, until their own first value of 52.5 in. (Method *A*) was arrived at early in March, 1914, to be confirmed before the end of that month by the short-time evaporation tests at Lake Conchos entering into Methods *B* and *C*, and in August by the results of the longer-time local evaporation tests.

It is of interest here to explain how Mr. Freeman arrived at his value of 55 in. in 1911, as stated by him in a letter of November 19th, 1914:

"With reference to the evaporation from Lake Conchos: My judgment was not based on experiments for the Medina Reservoir, Texas, as I note you infer; but really was based on the elaborate investigations that I made near Prattville, Cal., in 1907, in connection with my studies of the Great Western Power project. At that time I maintained for upward of 6 months what was probably one of the very best meteorological observatories on the Pacific Coast. I had among other instruments a recording hygrometer which we checked up three times daily with a sling psychrometer; and had also an evaporating tank carefully constructed in the midst of a pond, on which (after first noting the small evaporation at night) I had for a week or two continuous observations made throughout the 24 hours.

"My recording hygrometer (which was carefully checked) showed that very soon after dark the relative humidity came down close to the dew point and staid there until sunrise, thus completely checking evaporation.

"I noted at La Boquilla (Lake Conchos), during my 2 weeks on the ground, a somewhat similar chilling of the air at night. I also had in mind the Salton Sea experiments, showing the progressive diminution of evaporation over a large pond area, as compared with a good tank on shore. Moreover, I have noted in Arizona while driving by irrigated and non-irrigated lands at night, the distinct difference in the feeling of the air on the leeward side as one passed a freshly irrigated tract; which helped confirm me in the belief (first started by observations on the windward and leeward sides of the pond at Prattville, Cal.) that after the air has had its thirst partly appeased, it drinks up water less rapidly".

It appears to the writers that a value of evaporation depth arrived at by the judgment of an able engineer, of wide experience in that subject in similar regions, and after 2 weeks' observance of local conditions, is worthy of a better term than that of "guess" applied

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to it by Mr. Horton, even though Mr. Freeman's judgment was not aided by local tests of evaporation; and the writers do consider that the close agreement of his value with their own adds some corroborative weight to the latter.

As to "repeatedly calling attention to this guess"; Mr. Freeman's value is mentioned but twice in the paper, once on page 1845, in connection with Mr. Stearns' value, and again on page 1848, in connection with the writers'.

Mr. Hegly's Table 77 is of much interest as showing the comparatively uniform distribution of yearly evaporations among the 12 months, even for two such widely separated places (Lake Conchos, Mexico, and Arles, France) with presumably greatly differing climates. The same nearly uniform monthly distribution of evaporation is evident in Mr. Post's Table 62 (monthly distribution in Sweetwater Reservoir and in four pans, all near San Diego, Cal.); in the writers' Table 20 (Piche evaporimeter records at eleven places and pan records at five places, all in Texas and New Mexico), and is best shown in a general comparative table—Table 93.

To show the comparatively small differences at the different localities, Table 93 may be summarized as given in Table 94.

The departures from the means at the ten places included in Table 93 average + 2.8 and — 2.5%, with extremes for the separate months of + (5.2 to 1.6)% and — (1.5 to 4.0) per cent. These values of departures seem small in themselves, but are of considerable magnitude in comparison with the average monthly percentage of 8.3.

The ten localities were chosen at random, merely because for them the monthly percentages were available quickly and easily. Though the monthly percentage values at the ten places are quite uniform in a general way, and are most of them within rather narrow ranges, still in some instances they vary too much to permit the safe use of any general uniform values of monthly percentages.

Mr. Hegly's explanation of a greater proportion of the yearly evaporation occurring during the summer at Arles than at Lake Conchos* (because of the hotter and dryer summers at Arles) is interesting and probably true; since at Lake Conchos the rainy season, with its more humid conditions, occurs during the summer.

Mr. Hawgood's short discussion† is of much importance as calling attention to the Baldwin Latham experiments, showing the very great influence on the depths of pan evaporations of various elements (such as painted or unpainted pans, different colors with which the pans may be painted, etc., etc.) hardly ever recorded in the notes of evaporation tests. As bearing on such omissions, not only are all

* See Table 93, Columns (1) and (11).

† pp. 1946–48.

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TABLE 93.—COMPARATIVE MONTHLY PERCENTAGE DISTRIBUTION OF YEARLY EVAPORATION DEPTHS, AT TEN WIDELY SEPARATED LOCALITIES.

Month.	MONTHLY PERCENTAGE DISTRIBUTION OF YEARLY EVAPORATION DEPTH AT:											
	(a) Lake Conchos, near Chihuahua, Mexico.	(b) Fifteen places in Southern Texas and New Mexico.		(c) Sweetwater Reservoir, near San Diego, Southern California.	(d) Kingsburg, near Fresno, California.	(e) Santa Clara Valley, California (near San Jose).	(e) Lake Tahoe, Northern California.	(f) Chestnut Hill, Boston, Mass.	(f) Lee Bridge, England.	(g) Emdrup, Denmark.	(h) Arles, near Marseilles, France.	Mean of Columns (1) to (11).
		Mean of Piche records at eleven places.	Mean of pan records at five places.									
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
	%	%	%	%	%	%	%	%	%	%	%	%
May	11.5	10.0	12.2	12.0	9.7	10.0	8.0	9.6	13.0	13.8	11.4	11.0
June	11.7	11.6	13.8	15.0	13.5	14.0	11.0	13.9	15.3	19.0	13.3	13.8
July	12.2	12.4	12.8	13.0	16.4	17.0	14.0	15.7	16.4	18.5	17.1	15.0
August	11.1 46.5	11.6 45.6	13.1 51.9	10.0 50.0	17.7 57.3	16.0 57.0	16.0 49.0	15.1 54.3	13.8 58.5	15.6 66.9	14.3 56.1	14.0 53.8
September	9.4	9.0	10.9	9.0	13.6	14.0	13.0	12.7	7.9	9.3	9.5	10.8
October	7.7	8.3	7.5	5.0	7.7	7.0	10.0	8.7	4.9	4.5	6.7	7.1
November	5.1	7.0	4.8	4.0	4.3	4.0	7.0	5.8	3.5	2.5	3.8	4.7
December	4.1	4.9	3.2	4.0	2.3	2.0	4.0	3.5	3.0	1.8	2.9	3.3
January	4.8	4.5	2.9	5.0	1.6	2.0	4.0	2.4	3.7	2.5	2.9	3.3
February	5.6	5.5	2.9	5.0	2.6	2.0	3.0	2.8	3.2	1.8	3.8	3.5
March	7.4	7.0	7.2	7.0	5.0	5.0	4.0	3.6	5.1	3.6	6.7	5.6
April	9.4 53.5	8.2 54.4	8.7 48.1	11.0 50.0	5.6 42.7	7.0 43.0	6.0 51.0	6.2 45.7	10.2 41.5	7.1 33.1	7.6 43.9	7.9 46.2
Year	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

(a) Table 6, Table 77.
(b) Table 20.
(c) Tables 56 and 62.
(d) Computed from Table 68.
(e) *Engineering News*, Vol. 67, pp. 381–82 (February, 1912).
(f) Computed from Table 69.
(g) Fanning's "Water-Supply Engineering", 1906 (Table 24), p. 89.
(h) Table 77.

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TABLE 94.—MEAN OF MONTHLY PERCENTAGES OF YEARLY EVAPORATION DEPTH AT TEN LOCALITIES, WITH MAXIMUM DEPARTURES FROM MEAN.

Month.	Mean of monthly percentages.		Maximum departures from mean.			
May	11.0		+ 2.8%		and − 3.0%	
June	13.8		+ 5.2		and − 2.8	
July	15.0		+ 3.5		and − 2.8	
August	14.0	53.8%	+ 3.7	+ 13.1%	and − 4.0	− 8.2%
September	10.8		+ 3.2		and − 2.9	
October	7.1		+ 2.9		and − 2.6	
November	4.7		+ 2.3		and − 2.2	
December	3.3		+ 1.6		and − 1.5	
January	3.3		+ 1.7		and − 1.7	
February	3.5		+ 2.1		and − 1.7	
March	5.6		+ 1.8		and − 2.0	
April	7.9	46.2%	+ 3.1	+ 8.2%	and − 2.3	− 13.1%
Sums and Means	(8.3)	100.%	+ 2.8%	(+ 1.8%)	and − 2.5%	(− 1.8%)

such data of painting, color, material of pan, etc., practically always omitted from the published data of pan evaporations, but it is exceptional when even the depth of the pan and of its water are mentioned, and not at all unusual to find omitted the size of the pan, whether land or floating, etc. Many such omissions occurred in the published Texas and New Mexico evaporation data used by the writers in this problem; and but few of them could be supplied, even by considerable time and effort in correspondence. Probably most engineers who have dealt with evaporation problems have been impressed with both the general scarcity and the incompleteness of the published data.

As stated by Mr. Hawgood, there are many uncertainties in relative measurements of evaporations from pans and from reservoirs; and the utmost the writers ever have hoped for is the consideration of "their results and conclusions as a reasonable basis for reliable estimations", as stated by Mr. Hawgood.

As intimated already by the writers, they do not believe that any very useful results, of much practical value to engineers, can be attained by attempts to establish a "scientific formula" for evaporation depths, by which (from substitutions of the supposed evaporation effects of the various elements of a climate) reliable evaporation depths from reservoirs may be estimated synthetically.

Instead, they believe that useful advance in the estimation of evaporation losses is more likely to be in the direction of establishing gradually, in various scattered localities, an increasing number of fairly well founded values of evaporations from reservoirs them-

selves, from which (as a sort of primary network) evaporation values for contemplated intermediate reservoirs may be estimated by modification, by making comparative pan tests with the nearest primary reservoirs (supposedly under more or less similar climatic conditions), and working up the results of such pan comparisons by empirical and graphical methods, somewhat similar to those used by the writers in this paper. Messrs. Duryea and Haehl.

TABLE 95.—MEASURED YEARLY RESERVOIR EVAPORATIONS NEAR SAN DIEGO, CAL.

Reservoir.	YEARS' EVAPORATION, IN INCHES, FOR:						Means, in inches.
	1910.	1911.	1912.	1913.	1914.	1915.	
* Sweetwater (5 years)	59.5	63.3	57.5	57.8	45.8	.....	56.8
† Cuyamaca (4 years)	.....	.....	56.2	38.9	67.0	68.4	57.6
‡ Upper Otay (3 years)	.....	.....	.....	61.1	67.5	58.2	62.3
Means	59.5	63.3	56.85	52.6	60.1	63.3	(58.9) 59.3

* From Table 56.
† See explanation following Table.
‡ From Table 75.

As a beginning in the establishment of such a primary network of reasonably reliable values of evaporation from reservoirs themselves, it is believed that the data of measured reservoir evaporations in the vicinity of San Diego, Cal., contributed by Mr. Post* and Mr. Whitney† are of great and permanent value to engineers.

As mentioned by Mr. Hawgood,‡ measured evaporations from nearly all reservoirs are subject to the risk of more or less error because of the possibility of the loss of material quantities of the water by absorption and seepage; and it is realized that the San Diego data of Messrs. Post and Whitney are not without possibilities of such inaccuracies, and that, as presented, they include necessarily some assumptions and deductions. It is felt, however, that notwithstanding such possible inaccuracies, they will furnish by proper treatment a value for reservoir evaporation near San Diego, which may be regarded as fairly reliable and approximately correct.

All the measured yearly evaporation depths from reservoirs near San Diego, Cal., are summarized in Table 95.

* pp. 1906–18.
† pp. 1983–91.
‡ p. 1946.

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At the Cuyamaca Reservoir, reservoir evaporations were measured as follows:

Year	*1912....	June	to	September,	inclusive	(4 months)	= 28.12 in.
"	1913....	May	"	"	"	(5 "	= 23.73 "
"	1914....	March	"	"	"	(7 "	= 48.84 "
"	1915....	June	"	"	"	(4 "	= 34.22 "

From Tables 56 and 62, the monthly distribution of the yearly evaporation depth was as follows at the near-by Sweetwater Reservoir:

March	5%	
April	7	
May	11	
June	12	
July	15	
August	13	
September	10	73%
October	9	
November	5	
December	4	
January	4	
February	5	
Year	100%	

and differed but little from the monthly distributions at four evaporation pans in the vicinity.

Assuming the same monthly distributions at the Cuyamaca Reservoir as at the Sweetwater, the corresponding probable yearly depths of evaporation at the Cuyamaca would be as follows:

	Measured.	Year (deduced).
1912—4 months, June to September, inclusive	= 28.12 in. + 0.50 =	56.2 in.
1913—5 months, May to September, inclusive	= 23.73 in. + 0.61 =	38.9 in.
1914—7 months, March to September, inclusive	= 48.84 in. + 0.73 =	67.0 in.
1915—4 months, June to September, inclusive	= 34.22 in. + 0.50 =	68.4 in.
Mean yearly	=	57.6 in.
"Weighted" mean yearly	=	58.4 in.

If the "mean" values of Table 95 are weighted for the numbers of years (Sweetwater, 5 years, Cuyamaca, 4 years, and Upper Otay, 3 years), or for the numbers of reservoirs (1 for 1910, 1 for 1911, 2 for 1912, 3 each for 1913 and 1914, and 2 for 1915), then such weighted mean value is 58.4 in. with a "weight" of 12 reservoir-years.

Table 95 includes values of yearly evaporation depth for 6 different years; and at three separate reservoirs, which (even though all near San Diego) differ from each other considerably in altitude and in loca-

* From Table 54.

tion, and probably more or less in evaporation conditions. The altitudes of the three reservoirs are about as follows: Sweetwater 200 ft. above sea level, Upper Otay 500 ft., and Cuyamaca 4 600 ft.; their distances from the Pacific Coast are about as follows: Sweetwater 9 miles, Upper Otay 12 miles, and Cuyamaca 40 miles. Cuyamaca is about 32 miles from Sweetwater and about 31 miles from Upper Otay, and Sweetwater is about 6 miles from Upper Otay, and all the intervening lands are quite rough and mountainous. Messrs. Duryea and Haehl.

Because of the three different reservoirs and the six different years, and because each of the 12 values of Table 95 may be the correct measure of a year's evaporation depth for its own place and year, the values of that table are not amenable to rejection or acceptance by the test of Peirce's criterion; though, if that test were applicable, at least two of the values would be rejected (the 38.9 in. for Cuyamaca in 1913, and the 45.8 in. for Sweetwater in 1914). However, even though the law of probable error is not strictly applicable to Table 95, if applied to its 12 values, the result is a probable error (?) of its mean (58.4 in.) of $\pm$ 1.7 in.

Hence, in so far as the 3 reservoirs, the 6 years, and the 12 values of Table 95 may be supposed to represent a fair range of evaporation conditions in the vicinity of San Diego, the value of 58.4 in. per year (not less than 56.7 nor more than 60.1 in.) evaporation from reservoirs, may be accepted tentatively as the most probable general value at present for that region. This is always subject to correction and amendment if additional values are secured for other years and for other reservoirs.

Similarly, the mean of the 5 yearly values for Sweetwater Reservoir has a probable error (?) of $\pm$ 2.0 in., the mean of the 4 values for Cuyamaca has a probable error (?) of $\pm$ 4.6 in., and the mean of the 3 values for Upper Otay has a probable error (?) of $\pm$ 1.9 in.

The various present tentative values of yearly evaporation from reservoirs, near San Diego, may be summarized as in Table 96.

The Principal Conclusion of the paper (the probable yearly evaporation depth from Lake Conchos) is shown in some detail in Tables 3 to 6; and the finally adopted value is 55 in., before the deduction of the rain falling directly on the lake surface. The various discussions of the Principal Conclusion will be considered next.

Mr. Mathewson* does not discuss the Principal Conclusion; but some use of his discussion already has been made,† and a slightly different use will be made of it here, as follows:

The altitude and the mean yearly temperature of the Botello power-house are, respectively, about 6 560 ft. and 62.4° Fahr.,‡ and

* pp. 1902–03.

† See pp. 2041–43.

‡ Table 92 and Table 85.

Messrs. Duryea and Haehl.

those of Lake Conchos are about 4 300 ft. and 66.9° Fahr.* From Diagram (*b*), Fig. 4 (from the larger-scale original drawing, extended by straight lines to altitude 6 560 ft.), the monthly evaporation in a 3-ft. floating-pan is 8.88 in., at altitude 6 560 ft. and mean temperature 62.4° Fahr.; and at 4 300 ft. and 66.9° it is 6.98 in., or 1.90 in. per month or 22.8 in. per year less, as measured in a 3-ft. floating-pan. From Subsidiary Conclusion (*c*), the corresponding less depth as measured in a reservoir should be $(22.8 \times 0.62) = 14.1$ in. per year.

TABLE 96.—PRESENT TENTATIVE VALUES OF YEARLY EVAPORATION FROM RESERVOIRS, NEAR SAN DIEGO, CAL.

Reservoir.	Altitude, in feet.	Period, in years.	Probable Evaporation Depth per Year.	
			Mean, in inches.	Probable error of mean, in inches.
Sweetwater.....	200	5, 1910-14	56.8	± 2.0
Cuyamaca.....	4 600	4, 1912-15	57.6	± 4.6
Upper Otay.....	500	3, 1913-15	62.3	± 1.9
Near San Diego, Cal.....	*1 700	6, 1910-15	58.4	*± 1.7 (2.8 "weighted")

* If the "probable errors" of the three reservoirs are weighted in proportion to their numbers of years, the weighted average is ± 2.8 in., in comparison with the ± 1.7 in. of Table 96; and if the three altitudes are similarly weighted in proportion to their numbers of years, the resulting value (which may be called the "weighted average altitude") is about 1 700 ft.

From comparative local evaporation tests in a farm reservoir and in a small evaporation vessel, the La Botello engineers adopted 67.7 in.† as the value of yearly evaporation depth there from reservoirs; which, lessened by the estimated difference of 14.1 in., gives a corresponding yearly depth of evaporation from Lake Conchos of 53.6 in. This is 1.4 in., or 2.6%, less than the value of 55 in. adopted by the writers.

As admitted hereinbefore, this close check may be merely accidental; but, at the least, it does not weaken the validity of the writers' Principal Conclusion: that the yearly evaporation depth from Lake Conchos is about 55 in.

Mr. Chandler‡ refers to the writers' Principal Conclusion as follows:

"The conclusions drawn by the authors from this interesting and comprehensive investigation seem, in general, to be well founded and safely applicable in the region studied.

* * * * * * *

"* * * apparently the tables are not applicable with safety to places outside of the Great Plateau.

* * * * * * *

* Table 85.
† See p. 2042.
‡ pp. 1903–06.

"* * * if good records of evaporation are available, extending through a period of several years, at some point not remote from the region where estimates [of evaporation] are needed, it seems that the methods illustrated so clearly and completely in this paper will provide excellent means for making the corrections needful for the differences in evaporation, caused by comparatively small differences in elevation and in mean temperature; but these methods are not suited for the transfer of figures from actual records to far distant regions, or to points where the mean temperature or elevation is, in great degree, different." Messrs. Duryea and Haehl.

The writers cannot but agree, in general, with these remarks by Mr. Chandler.

Mr. Lee* discusses the writers' Principal Conclusion as follows:

"The authors have attempted, through assumptions and highly theoretical methods, carried out in great detail, to estimate annual evaporation from a lake surface more than 300 miles from the nearest point of actual observation. They have also attempted an exhaustive investigation and comprehensive analysis of the problem. They have based much of their framework, however, on a fundamental assumption† which the writer believes to be in error. As a result thereof, the computed value of yearly gross evaporation from Lake Conchos, as set forth in Table 4, is far from the truth."

It is noticeable that the writers' methods, characterized by Mr. Lee as "highly theoretical", are instead characterized by Mr. Horton‡ "as almost wholly empirical".

Mr. Lee is hardly correct in stating "the authors have attempted * * * to estimate annual evaporation from a lake surface more than 300 miles from the nearest point of actual observation". As a matter of fact, though his comment is partly true of Method *A*, no reliance was placed on the result by that method, and no use was made of it, until it apparently was confirmed by the results of Methods *B* and *C*, both based on observations of evaporation made at Lake Conchos itself.

Apparently, Mr. Lee dissents from the writers' Principal Conclusion chiefly because of his dissent from Subsidiary Conclusion (*c*), he believing that the evaporation depth from a floating-pan is about the same as (and at least not materially greater than) that from the reservoir on which it is floating. This question has been discussed hereinbefore more or less fully,§ with the writers' conclusion there that ‖"Subsidiary Conclusion (*c*) may be given at least tentative acceptance as expressive of the best data as yet available". Mr. Lee describes¶ some comparative evaporation tests made by him on Owens

* p. 1921.
† Subsidiary Conclusion (*c*) = 0.62.
‡ p. 1997.
§ See pp. 2024–33.
‖ See p. 2033.
¶ p. 1926.

Messrs. Duryea and Haehl.

River and Lake, which seem to prove that the evaporations from the lake and from a pan floating on the river were the same, and hence to disprove (*c*); but the pan was about 20 miles from the lake, which may make the comparison somewhat doubtful, though he states that lake and pan have "practically the same climatic conditions".

In his reference to Method *C*, Table 46, and Piche evaporimeter computations, Mr. Lee is in error. Table 31 (from which Table 46 statedly is derived) is based only on evaporations observed from "3-ft. square floating-pans". However, it is seen from Table 20 that, in respect to the monthly percentage distributions of the yearly evaporation, apparently pan distributions and Piche distributions vary but little from each other.

Also, though Mr. Lee objects to the writers' expansions by Methods *B* and *C* from 2 to 7 months to a full year, and apparently has some ground for his objection, it is noticeable that all the yearly evaporations thus estimated by expansion (by B_a and B_b from 2 and 5 months,* and by C_a and C_b from 4 and 7 months*) agree quite closely with one another, also with the yearly evaporation depth estimated by Method *A*,† which is independent of any method of expansion.

Referring to Mr. Lee's lack of confidence in Method *C*: the writers must acknowledge that the possible lack of sufficient precision, in the stream gauging rating curves and perhaps in some of the other measurements, may make the resulting values of the lake evaporation perhaps less accurate than they were thought to be, and that there may be some doubt (as suggested by Mr. Hawgood‡ and Mr. Comstock§) whether the average pan evaporation from such a large lake is represented closely by measurements in only one or two pans.

However, all measurements, of both pans and lake, were made with much care (probably with more care than generally is used for that class of work), the results of the two methods of working up the test observations for the two periods all agree with each other closely,‖ and it is felt that the probability of the yearly evaporation depths thus arrived at by Method *C* being seriously in error is not great. Also, in view of Professor Bigelow's deductions at Salton Sea (or at least of the writers' interpretation of them¶), of Mr. Post's results secured at San Diego,¶ and of those secured by the writers at Lake Conchos,¶ and because a reasonable cause has been suggested therefor,** it is felt that, on the whole (even though the possibility of error is acknowledged), there is every probability that the evaporation from a reservoir

* Tables 48 and 49.
† Tables 3 and 30 to 33.
‡ p. 1946.
§ pp. 1966–67.
‖ Tables 48 to 50.
¶ All discussed on pp. 2024–2033.
** See p. 2026.

is materially less than that from a 3-ft. pan floating thereon; and that the most probable value as yet (worthy of at least tentative acceptance) is about 62%, that used in Subsidiary Conclusion (*c*). Messrs. Duryea and Haehl.

Much of the foregoing discussion of Mr. Lee's contribution will apply also to that of Mr. Comstock.*

Referring to Mr. Comstock's statement that he †"has calculated the percentage distribution of evaporation at thirty stations in the United States, and has taken the mean for each month. The result shows 46% of the annual evaporation for the months, January to June, inclusive" (instead of the 50.4%‡ used by the writers): the 50.4% is derived from observed mean monthly temperatures at Lake Conchos itself,§ which monthly distributions (thus estimated) bear a close resemblance to the monthly distributions in Texas and New Mexico.‖ The 50.4% thus estimated seems more worthy of confidence than the 46% estimated "at thirty stations in the United States", especially as Table 93 shows that monthly percentage distributions of yearly evaporations, while not differing greatly from each other at different places, still do differ enough to make it quite advisable to compare only with places as near as practicable and having similar climatic characteristics.

Referring to the writers' Principal Conclusion (that the probable annual evaporation depth from Lake Conchos is about 55 in.), Mr. Grunsky says:

¶"On the assumption that the mean monthly temperatures [at Lake Conchos] are about as shown in Table 30, * * * the Kingsburg curve [Fig. 28] indicates a mean annual evaporation from the water surface of Lake Conchos of 60 in. This is subject to correction for higher elevation. (Lake Conchos is at Elevation 4 300 and Kingsburg is about at Elevation 300.) This correction is perhaps but little in the hot summer months, but may affect the results materially in the cold winter months. It is believed that this correction will not be as great as the curves presented in the paper would indicate, but, nevertheless, it may be an increase of from 20 to 30 per cent. It is the writer's opinion that further observations at Lake Conchos will show that the evaporation there will be between the 60 in. indicated by the Kingsburg curve and the 85 in. which the authors have estimated as the annual evaporation from a 3-ft. square pan floating on the water surface of the lake.

"The results at Kingsburg, though not recommended as fully dependable, have been tested by the fall of the water surface at Salton Sea, where the check was satisfactory for the year 1907-1908."

Mr. Grunsky's opinion—that the mean annual evaporation from the surface of Lake Conchos is between 60 and 85 in. (instead of being the 55 in. found by the writers)—apparently is based on an assumption

* pp. 1966–67.
† p. 1966.
‡ p. 1894.
§ Table 31.
‖ Fig. 11.
¶ p. 1982.

Messrs. Duryea and Haehl.

that the climates of Kingsburg and Lake Conchos differ but little from each other except in mean temperatures and in altitudes. This, however, is not the fact. As pointed out by Mr. Jorgensen,

*"At Lake Conchos, fortunately, the rainy season coincides with the season of maximum temperature, and this condition undoubtedly tends to keep down evaporation loss; otherwise one would expect the yearly evaporation depths there to be higher than 55 in., due to the high average temperature and the very irregular shape of the lake."

At Kingsburg the rainy season (with its higher humidity) occurs in the winter, when the mean temperatures and the monthly evaporations are the least; at Lake Conchos, Mexico, it occurs in the summer. Thus Lake Conchos has higher relative humidities and lower relative mean temperatures and monthly evaporations throughout the summer, than Kingsburg. Hence it is believed that Mr. Grunsky is not justified in estimating a Lake Conchos evaporation value from his Kingsburg curve, with no other corrections except that for the difference in the altitudes; and that any Lake Conchos evaporation thus estimated necessarily must be greater than the true value.

Also opposed to Mr. Grunsky's opinion (that the probable mean annual evaporation depth from Lake Conchos is between 60 and 85 in.) is the opinion of Mr. Freeman, who, after 2 weeks spent there in 1911 in study of the La Boquilla project, reported that in his opinion the annual evaporation from the lake would be about 55 in.;† and the value of 53.6 in. for Lake Conchos deduced by the writers‡ with the aid of Fig. 4 from a carefully estimated value of reservoir evaporation at the Botello power-house. The Botello power-house is about 600 miles south of Lake Conchos, but still within the limits of the Great Plateau, and presumably with climatic characteristics quite similar to those at Lake Conchos.

This close check on the writers' Principal Conclusion of 55 in. possibly is merely an accidental coincidence; but, unless such, it adds at least some corroborative weight to the validity of their Principal Conclusion, of their Subsidiary Conclusion (*c*) (that the evaporation depth from a reservoir is only about 62% of that from a 3-ft. pan floating thereon), and to the proper applicability of Fig. 4 to at least one other locality on the Great Plateau, at great distances from both Lake Conchos and Southern Texas and New Mexico.

The paper and its discussions include annual evaporation depths from reservoirs (with corresponding altitudes and mean temperatures) not only at Lake Conchos and at the Botello power-house, but also at the Salton Sea (estimated by Mr. Grunsky and others) and at the Cuyamaca Reservoir (measured by Mr. Post). Merely from

* p. 1932.
† See pp. 2045–46.
‡ See pp. 2051–52.

PLATE LXII.
TRANS. AM. SOC. CIV. ENGRS.
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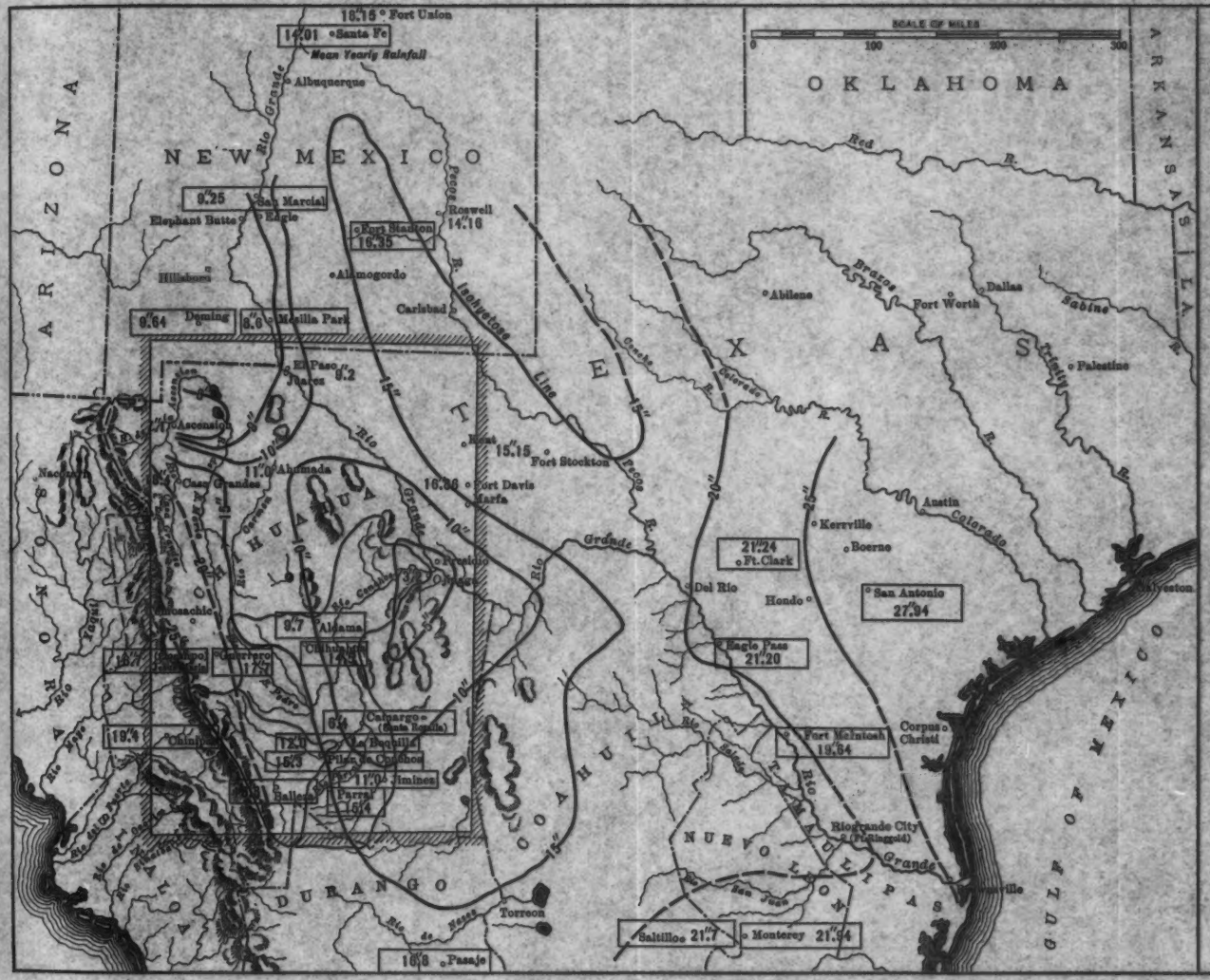
DURYEA AND HAEHL ON
EVAPORATION FROM LAKE CONCHOS, MEXICO.


SCALE OF MILES
0
100
200
300
Mean Yearly Rainfall
16.15 Fort Union
14.01 Santa Fe
Albuquerque
NEW MEXICO
ARIZONA
OKLAHOMA
ARKANSAS
Red R.
Rio Grande
Pecos
8.25 San Marcial
Elephant Butte
Eagle
Roswell 14.16
Fort Stanton 16.35
Hillsboro
Alamogordo
Carlsbad
9.64 Deming
8.0 Mesilla Park
El Paso 9.2
Isohyetose Line
Abilene
Brazos
Fort Worth
Dallas
Sabine
Trinity
Palestine
Colorado
Concho R.
Ascension
Nacimiento
Casas Grandes
Ahumada
Rio Conchos
Rio Florido
Rio Chuviscar
Rio Sacramento
Rio Parral
Rio Nazas
Rio Aguanaval
Rio Mezquital
Rio Tunal
Rio Nadadores
Rio Salado
Rio Sabinas
Rio Alamo
Rio San Juan
Rio Salinas
Rio Pilon
Rio Blanco
Rio Verde
Rio Santa Maria
Rio Grande

curiosity, Fig. 4 was applied to the Salton Sea and to the Cuyamaca Reservoir; and in both instances the resulting annual evaporations by the diagram from 3-ft. floating-pans gave (when multiplied by the 62% value of (*c*)) an estimated value of the annual evaporation depth from the lake or reservoir which was much smaller than the evaporation locally estimated or measured. This is not surprising, however, as both Salton Sea and Cuyamaca Reservoir are far removed from the Great Plateau and its conditions, and presumably with very dissimilar climatic characteristics. Messrs. Duryea and Haehl.

In determining the probable power production of the La Boquilla project, a careful study was made by the writers of the rainfall and stream flow of the State of Chihuahua and parts of Northern Mexico; and the most important of the data acquired during that study are given here as contributions toward future hydrographic investigations of that region.

The rainfall data thus offered are given in the form of the isohyetose map, Plate LXIX; and as Tables 79 to 82, 90, and 91, and parts of Tables 97 and 98. The stream flow data are given as Tables 83 and

TABLE 97.—MEASURED TOTAL STREAM FLOW AND DEDUCED AVERAGE YEARLY RAINFALL, OF THE ENTIRE DRAINAGE AREA OF THE RIO CONCHOS ABOVE ITS MOUTH, FOR EACH OF THE 14 YEARS, 1900-1913.

(This drainage area is very nearly 23 000 sq. miles.)

Year.	AVERAGE RAINFALL ON AREA:		MEASURED YEARLY STREAM FLOW OF RIO CONCHOS AT ITS MOUTH:		
	Year's rainfall as a percentage of the mean rainfall.	Year's rainfall, average depth, in inches.	As a uniform depth, in inches, on drainage area.	As a percentage of the yearly rainfall.	As cubic feet per second per square mile of drainage area.
(1)	(2)	(3)	(4)	(5)	(6)
1900	(123)*	17.6	0.86	4.9	0.0633
1901	80	11.4	0.32	2.8	0.0236
1902	97	13.9	1.20	8.6	0.0885
1903	93	13.3	0.54	4.1	0.0398
1904	110	15.7	1.84	11.7	0.1355
1905	152	21.7	2.02	9.3	0.1487
1906	126	18.0	2.31	12.8	0.1703
1907	96	13.7	1.08	7.9	0.0796
1908	84	12.0	0.79	6.6	0.0582
1909	85	12.2	1.10	9.0	0.0810
1910	50	7.2	0.47	6.5	0.0346
1911	122	17.4	1.48	8.5	0.1090
1912	(91)*	13.0	1.18	9.1	0.0870
1913	(113)*	16.2	0.73	4.5	0.0538
Averages	(101.5) 101.5	14.52†	1.14	7.86	(0.0838) 0.0840

* The rainfalls of 1900, 1912, and 1913 are less reliable than those of the other years, because based almost wholly on American rainfall data.

† The deduced mean yearly area-rainfall for a long period (62 years) is 14.3 in.

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TABLE 98.—MEASURED TOTAL STREAM FLOW AND DEDUCED AVERAGE YEARLY RAINFALL, OF THE ENTIRE DRAINAGE AREA OF THE RIO CONCHOS ABOVE LA BOQUILLA, FOR EACH OF THE 6 YEARS, 1910-1915.

(This drainage area is very nearly 7 000 sq. miles.)

Year.	AVERAGE RAINFALL ON AREA:		MEASURED YEARLY STREAM FLOW OF RIO CONCHOS AT LA BOQUILLA:		
	Year's rainfall as a percentage of the mean rainfall.	Year's rainfall, average depth, in inches.	As a uniform depth, in inches, on drainage area.	As a percentage of the yearly rainfall.	As cubic feet per second per square mile of drainage area.
1910.....	50	9.6	1.29	13.5	0.0955
1911.....	122	23.4	2.28	9.8	0.1680
1912.....	91	17.5	2.05	11.7	0.1515
1913.....	113	21.7	1.48	6.8	0.1088
1914.....	(128)*	(24.6)*	5.29	21.5	0.3890
1915.....	(131)*	(25.2)*	2.11	8.4	0.1552
Averages..	(105.8)	(20.3)	(2.42)	(12.0)	(0.1780)
	.....	19.2†	2.17†	12.6†	0.160†

* Less accurate than other years, only two rainfall stations.

† Deduced means for long period, 62 years.

84 and parts of Tables 97 and 98. In Tables 85, 86, and 92 are given data of mean, mean maximum, and mean minimum, temperatures; in Tables 93 and 94, data of the monthly percentage distributions of the yearly evaporation, at several places; in Tables 95 and 96, summarized data of the measured reservoir-evaporations near San Diego, Cal.; and in Table 99, data of maximum flood rates of the Rio Conchos at La Boquilla.

Rainfall Data.—The isohyetose map, Plate LXIX, was constructed from the mean yearly rainfalls of Tables 79 and 82 and some few additional data, and is believed to be fairly reliable in a broad sense for such a map, with a tendency to too low rather than to too high mean yearly rainfalls.

Stream Flow Data.—As explained already, the flows of the Rio Conchos were measured at its mouth for the 14 years, 1900-1913, inclusive. The measurements there were discontinued early in 1914. Its flow was measured also at Lake Conchos for the 6 years, 1910-1915, inclusive, and the measurements there are being continued.

The flows measured at the mouth of the Rio Conchos are given in Table 97 and those at Lake Conchos in Table 98.

Columns (3) and (5) of Table 97 were plotted on co-ordinate paper, and upper and lower envelopes drawn of twelve of the fourteen points. The corresponding equations of the upper and lower envelopes and of the "median" (not far from the "mean") line are as follows:

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$$\left.\begin{array}{l}\text{Year's stream flow as a percentage of the year's rainfall}\end{array}\right\} = \left\{\begin{array}{l}\text{Upper envelope—} \\ \quad [(\text{Inches of rainfall} \times 0.633) + 2.2]\% \\ \text{Median line—} \\ \quad [(\text{Inches of rainfall} \times 0.633) - 1.3]\% \\ \text{Lower envelope—} \\ \quad [(\text{Inches of rainfall} \times 0.633) - 4.8]\%\end{array}\right.$$

Year's Stream Flow at La Boquilla, as a Percentage of That at the Mouth of the Rio Conchos.—Stream flows were measured at both places for each of the 4 years 1910-13, with some variations in the accuracy and reliability of the Lake Conchos measurements in the different years. A comparison of the measured flow at La Boquilla each year, with the flow at the mouth of the river in the same year, gave the following relative flows:

1910—Total flow at La Boquilla = 86% of that at mouth.
1911— " " " " " = 47% " " " "
1912— " " " " " = 54% " " " "
1913— " " " " " = 62% " " " "

The most accurate of the years' measurements of stream flow at La Boquilla is believed to be that for 1912, with those for 1910 and 1911 less accurate and reliable than the others.

The year of lowest total stream flow at both places was 1910, with 1913 next higher, 1912 still higher, and 1911 highest. Hence it appears probable (as would be expected) that in years of low total stream flow the total flow at Lake Conchos is a greater percentage of the total flow at the mouth of the river, than it is in years of high stream flow.

A careful comparative study was made of the cumulative monthly stream flows at the two places for periods of 12, 24, 36, and 48 months, both with the years in their natural chronological order and re-arranged in the order of their supposed accuracies. Considered in either way, it was found that at the end of 24 months the cumulative stream flow at La Boquilla was 57% of that at the mouth of the river, the same at the end of 48 months, and not materially different at the end of 36 months. As a final conclusion, the flow of the Rio Conchos at La Boquilla (from its about 7 000 sq. miles of drainage area) was adopted as a straight 55% of that at the mouth of the river (from its about 23 000 sq. miles of total drainage area).

Hence it is believed that the intensity of the total yearly stream flow from the upper 7 000 sq. miles of the Rio Conchos drainage area is about $\left(\frac{55}{7\,000} \div \frac{100}{23\,000}\right) = 1.8$ times as great as the average intensity from the total, including drainage area (above the mouth of the river) of about 23 000 sq. miles.

Messrs. Duryea and Haehl.

Probable Maximum Flood Flows from Lake Conchos.—In 1911 Mr. Freeman made as careful an examination as practicable of former maximum floods of the Rio Conchos at La Boquilla, basing his study on a few gaugings of the river and on the flood marks of past years as pointed out by "oldest inhabitants". The results of his study are given in Table 99.

TABLE 99.—PROBABLE MAXIMUM FLOOD RATES OF THE RIO CONCHOS AT LA BOQUILLA, MEXICO, AS DEDUCED IN 1911 BY JOHN R. FREEMAN, M. AM. SOC. C. E.

Flood of year.	Probable maximum total flood flows (at crest of flood), in cubic feet per second.	Probable maximum flood flows per square mile of tributary drainage area, based on the most probable area of 7 100 sq. miles, in cubic feet per second per square mile.
1904	161 000	22.6
1868	230 000	32.6
1829	335 000	47.2
Usual *	134 000	18.9

* Probable extreme flood height of ordinary year.

Before Mr. Freeman's investigation, there had been adopted a spillway capacity of 2 000 cu. m. per sec. (70 600 cu. ft. per sec. total, or 10.0 cu. ft. per sec. per sq. mile). As a result of his investigation, a spillway capacity was adopted of 10 000 cu. m. per sec. (353 000 cu. ft. per sec. total, or 49.8 cu. ft. per sec. per sq. mile); and the spillway dam was constructed to carry that flow, with a depth of water of 3 m. (9.84 ft.) on its crest.

In closing this discussion, the writers (though acknowledging some inaccuracies and defects in the paper, pointed out in the discussions) still feel that they may claim for it that, in the words of Mr. Horton,

"* * * the work done in attempting a correlation of evaporation with temperature and altitude represents a start in a much needed investigation, and contains suggestions which may be useful in solving the vexed problem of the relation of evaporation to altitude."

Also, they feel that they may claim some permanent value for the paper because of the data it contributes to the rather meager knowledge of evaporation, rainfall, stream flow, and temperature in Northern Mexico—knowledge of much importance in connection with probable future hydraulic developments in that region.

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Paper No. 1377

THE DESIGN OF A DRIFT BARRIER ACROSS WHITE RIVER, NEAR AUBURN, WASHINGTON*

By H. H. Wolff, M. Am. Soc. C. E.

With Discussion by Messrs. H. M. Chittenden and W. J. Roberts.

Synopsis.

The purpose of this paper is to describe a structure, believed to be unique, for eliminating, from the upper reaches of a mountain stream, large quantities of drift, the accumulation of which in the lowlands had been causing much damage.

Before beginning a description of the drift barrier, a short outline of the project, of which it is a part, and the conditions which made its construction necessary, may not be amiss.

White River is one of the mountain streams of Western Washington, and derives its waters in part from the Cascades and in part from the glaciers of Mount Rainier. It runs to tide-water in Puget Sound, and in flood period is an exceptionally violent torrent, even for a mountain stream. The lower reaches of the river pass through a number of villages and through land of the highest agricultural value, and most of this land, on account of its sedimentary or alluvial nature, offers

* Presented at the meeting of May 3d, 1916.

very little resistance to erosion. The upper portion runs through very heavy timber, and in times of flood the water erodes the banks and carries with it large numbers of trees, saw-logs, stumps, and down timber, as drift.

The damage done in the past and the danger of greater damage in the future induced the authorities of King and Pierce Counties to undertake jointly the control and rectification of the river. The main features of the project include straightening the channel by dredging and otherwise, bank protection of various types, and a drift barrier. The work is being done under W. J. Roberts, M. Am. Soc. C. E., as Chief Engineer.

It will be seen from Fig. 1 that the river has deviated greatly from the shortest course, the normal bed, and in places has covered considerable areas, cutting away the land and leaving gravel bars below flood level. A close study of conditions shows that in every instance the current was first deflected by an accumulation of drift, the huge timber of this section serving readily in its formation.

Referring to the barrier site, where the river grade is 35 ft. per mile, drift begins running at about a 3-ft. stage of water. At the highest known stage, 9 ft., the quantity was very great, and the erosion of the banks, even in the upper reaches, very rapid, so that stumps from logged-off sections and full-grown trees were carried away. When one of these catches on an obstruction below, it quickly entangles others, and the mass of drift thus formed is solid enough to deflect the current.

Gravel, sand, and silt collect in the dead water, behind the drift piles, strengthening them and preventing the river from returning to its original bed. Evidences of this action are plentiful, and, in the narrow valley of the upper reaches, show that the river has been forced from the hills on one side to those of the other, a distance of ½ mile or more, and the original bed has become overgrown with very heavy timber.

It became essential, therefore, to erect some structure above the limit of the protective works and channel changes to eliminate or hold the drift and, after clearing the river from all drift lodged below or scattered along its bottom, prevent the accumulation of any more masses. The writer was entrusted by Mr. Roberts with the design and construction of such a barrier.

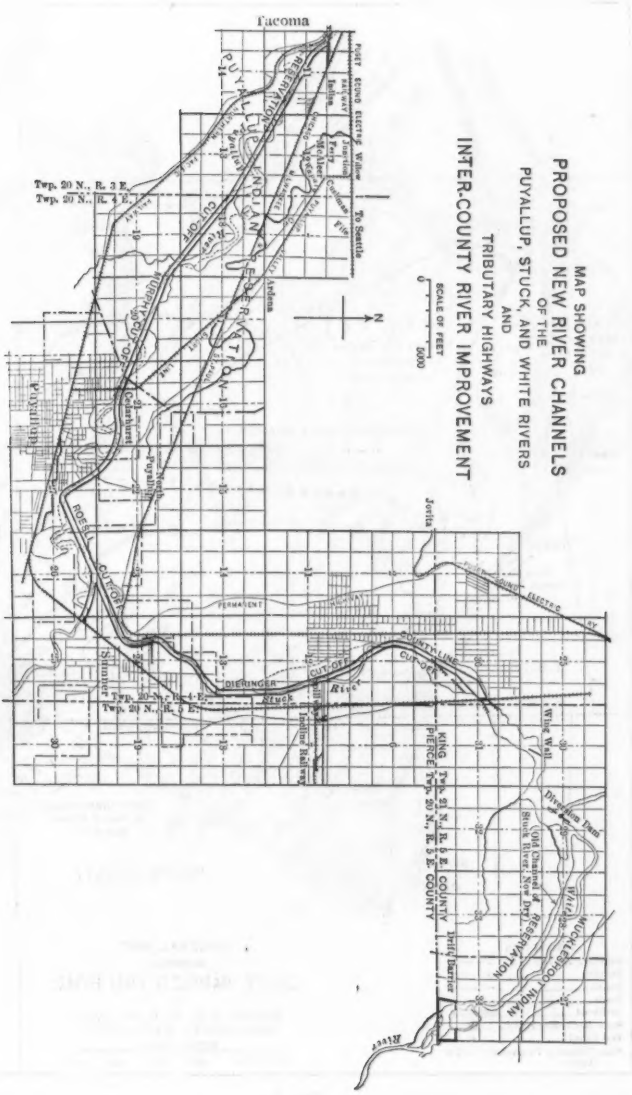

MAP SHOWING
PROPOSED NEW RIVER CHANNELS
OF THE
PUYALLUP, STUCK, AND WHITE RIVERS
AND
TRIBUTARY HIGHWAYS
INTER-COUNTY RIVER IMPROVEMENT
SCALE OF FEET
5000
Tacoma
Puyallup
Sumner
Orting
Kent
Auburn
Renton
Seattle
Edgewood
Milton
Fife
Dieringer
Alderton
Jovita
Twp. 20 N., R. 3 E.
Twp. 20 N., R. 4 E.
Twp. 21 N., R. 3 E.
Twp. 21 N., R. 4 E.
Twp. 20 N., R. 5 E.
KING
PIERCE
COUNTY
Cut-off
Diversion Dam
Drift Barrier
Muckleshoot Indian Reservation
White River
Stuck River
Green River
Puyallup River
Puyallup Indian Reservation
Permanent
Temporary
Old Channel of Stuck River, Now Dry
To Seattle

FIG. 1.

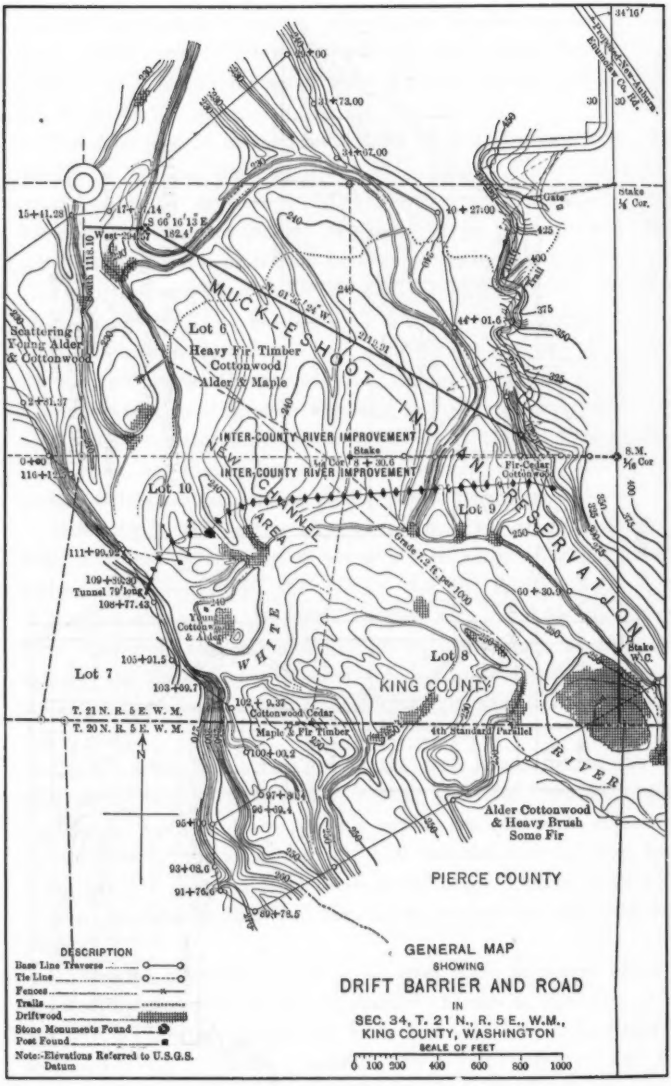

MUCKLESHOOT INDIAN RESERVATION
INTER-COUNTY RIVER IMPROVEMENT
NEW CHANNEL AREA
WHITE RIVER
KING COUNTY
PIERCE COUNTY
Lot 6
Heavy Fir Timber
Cottonwood
Alder & Maple
Lot 7
Lot 8
Lot 9
Lot 10
Alder Cottonwood & Heavy Brush Some Fir
Scattering Young Alder & Cottonwood
Maple & Fir Timber
Cottonwood Cedar
T. 21 N. R. 5 E. W. M.
T. 20 N. R. 5 E. W. M.
4th Standard Parallel
Tunnel 79' long
DESCRIPTION
Base Line Traverse
Tie Line
Fences
Trails
Driftwood
Stone Monuments Found
Post Found
Note:-Elevations Referred to U.S.G.S. Datum
GENERAL MAP
SHOWING
DRIFT BARRIER AND ROAD
IN
SEC. 34, T. 21 N., R. 5 E., W.M.,
KING COUNTY, WASHINGTON
SCALE OF FEET
0 100 200 400 600 800 1000

FIG. 2.

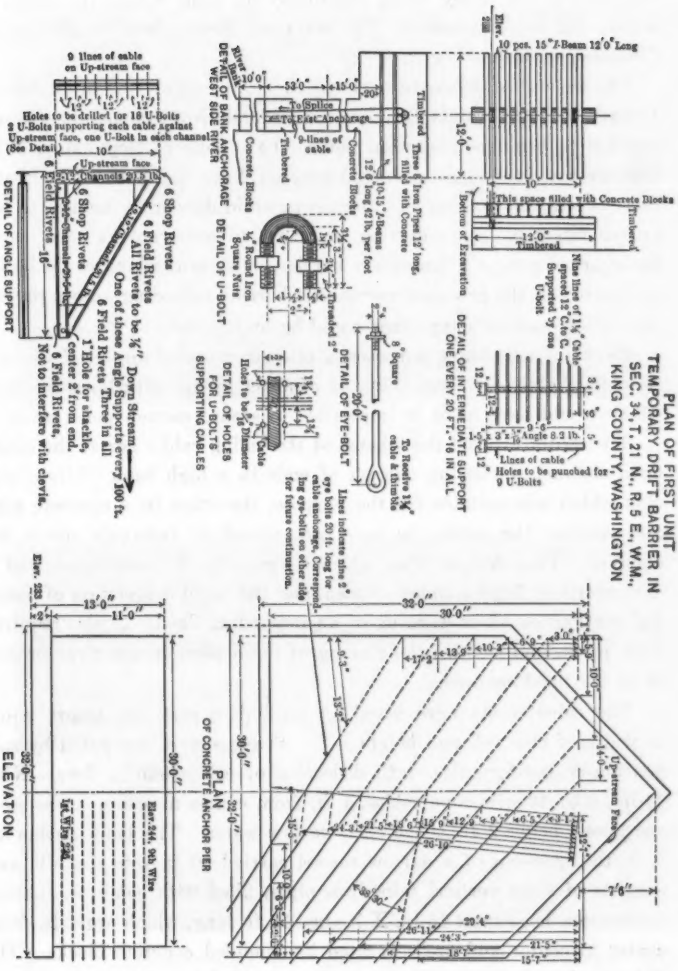

PLAN OF FIRST UNIT
TEMPORARY DRIFT BARRIER IN
SEC. 34, T. 21 N., R. 5 E., W.M.,
KING COUNTY, WASHINGTON
10 pcs. 15" I-Beam 12'0" Long
Timbered
Bottom of Excavation
This space filled with Concrete Blocks
Nine lines of 1¼" Cable spaced 12" C. to C. Supported by one U-bolt
DETAIL OF INTERMEDIATE SUPPORT ONE EVERY 20 FT.-16 IN ALL
Lines of cable
Holes to be punched for 9 U-Bolts
9 lines of cable on Up-stream face
Holes to be drilled for 10 U-Bolts at U-Bolts supporting each cable against Up-stream face, one U-Bolt in each channel (See Detail)
Up-stream face
Cap Channels 9" x 13"
6 Field Rivets
6 Shop Rivets
6 Field Rivets
All Rivets to be ¾" Down Stream
One of these Angle Supports every 100 ft.
6 Field Rivets Three in all
1" Hole for shackle, center 2" from end.
6 Field Rivets, Not to interfere with clevis.
DETAIL OF ANGLE SUPPORT
DETAIL OF BANK ANCHORAGE WEST SIDE RIVER
To Splice
To East Anchorage
9 lines of cable
Concrete Blocks
Timbered
Three I-Irons Piped 12' long, filled with Concrete
12'0" long 40 lb. per foot
DETAIL OF U-BOLT
Square Nuts
Round Iron
Threaded 2"
DETAIL OF EYE-BOLT
To Receive 1¼" cable & thimble
Holes to be ⅞" Diameter
DETAIL OF HOLES FOR U-BOLTS SUPPORTING CABLES
Lines indicate nine 2" eye bolts 20 ft. long for cable anchorages. Corresponding eye-bolts on other side for future continuation.
ELEVATION
PLAN OF CONCRETE ANCHOR PIER
Up-stream Face

FIG. 3.

Choice of site was determined by shortness of distance across the valley and nearness to the protective work farthest up stream. The character of the valley, being practically the same within the limits of choice, did not influence it. The site is an 80-acre tract bought by the Counties, shown on Fig. 2.

On account of delays in securing the land—which was in an Indian Reservation and bought through the Federal Government—it was impossible to begin construction before the middle of September, 1914. The severest floods have occurred between November 1st and 15th, and therefore it was imperative, in order to avoid danger of damage to improvements then being carried on below, to construct at once across the channel proper a temporary barrier, or a unit, which could be incorporated in the proposed permanent barrier extending across the entire valley, and of a type that could be built quickly.

Even for temporary purposes, a pile structure of any kind was not feasible, the river bottom being of compact gravel with large boulders which would have made it impossible to secure enough penetration.

The writer evolved the scheme of stretching cables across the entire flood channel, anchoring one set of ends in a high bank of indurated clay which was suitable for the purpose, the other in a concrete pier, and spacing the cables by spreaders placed at intervals across the channel. This design (Fig. 3) made possible the employment of a comparatively large number of men, and the rapid completion of essential portions on short warning of an impending flood. It also required little plant, and avoided the placing of main piers in the river proper, or in the flood channel.

The calculations were based on solid dam pressure, length equal to width of channel, and height 8 ft. Stresses were computed by suspension-bridge formulas, with deflection of cables one in four. Nine 1¼-in., 6 by 19 wire-rope cables, 1 ft. from center to center, were used, the lowest being 2 ft. above extreme low water. The bank anchorage is in the cross-cut of a narrow tunnel carried 80 ft. into the hill, and consists of three vertical 8-in. iron pipes, filled with concrete, bearing against ten horizontal 15-in. I-beams, 12 ft. long, which are 1 ft. from center to center and are separated by moulded concrete blocks. The I-beams bear against and are partly embedded in a block of concrete, filling a 2-ft. space between the I-beams and the wall and extending 13 ft. farther down the tunnel. The cables pass through a set of holes

FIG. 4.—GENERAL VIEW OF FIRST UNIT LOOKING WEST DURING 3-FOOT STAGE OF WATER.

FIG. 5.—CONCRETE SUPPORT FOR TRIANGLES, FIRST UNIT, ANCHORAGE BLOCK IN BACKGROUND.

LOOKING WEST.

INDICATING SIZE.

FIG. 6.—VIEW OF DRIFT BARRIER EXTENSION, LOOKING WEST.

FIG. 7.—TYPICAL PIER, DRIFT BARRIER EXTENSION, INDICATING SIZE.

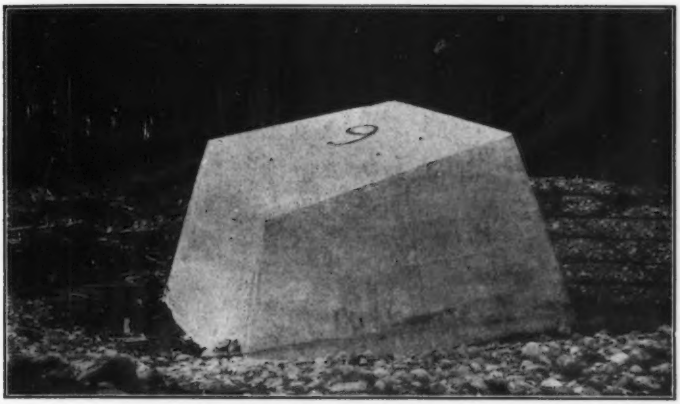


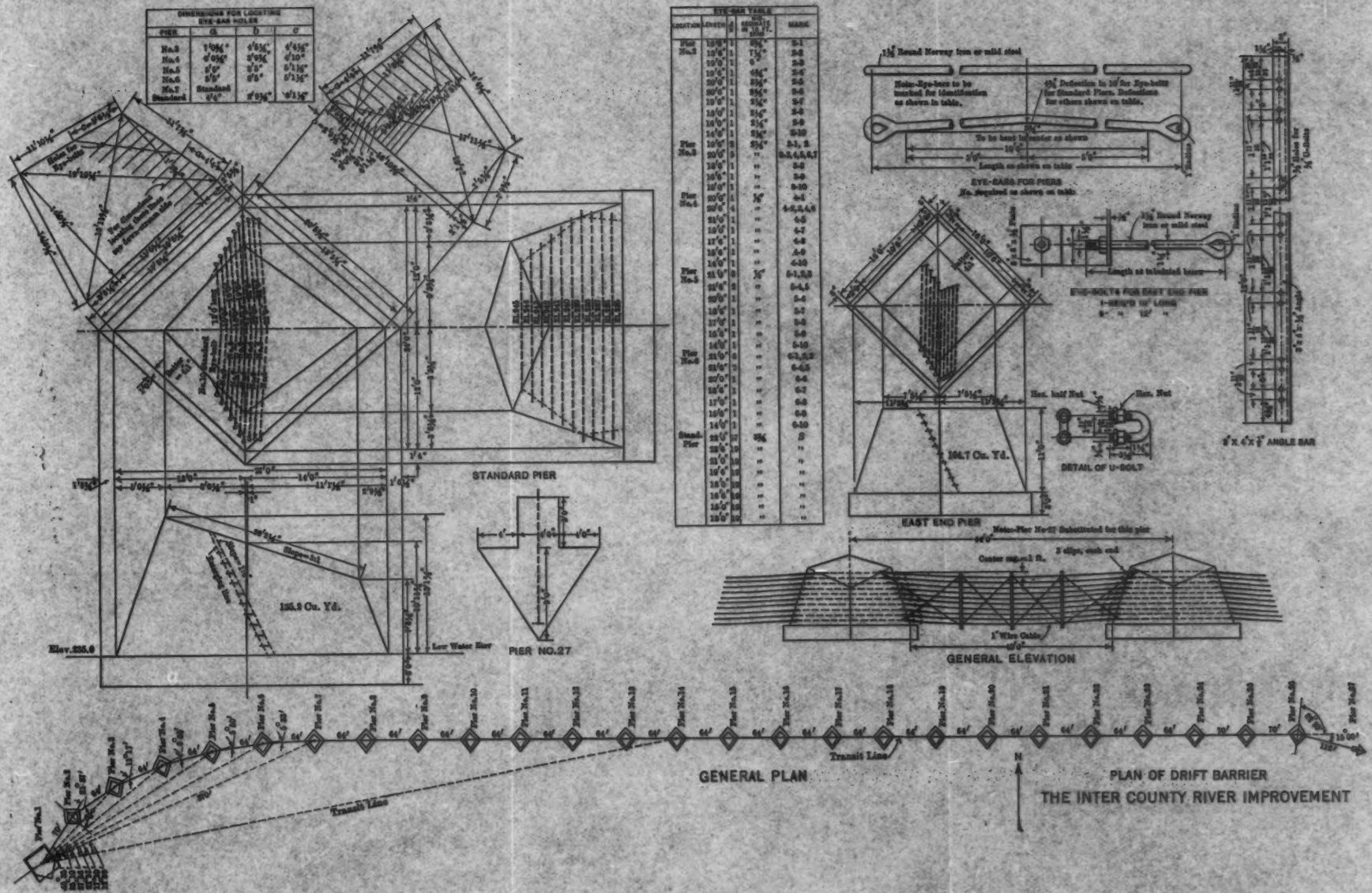
FIG. 8.—TYPICAL PIER, DRIFT BARRIER EXTENSION.

FIG. 9.—INTERMEDIATE WORK, DRIFT BARRIER EXTENSION.

FIG. 10.—NEW CHANNEL BETWEEN PIERS NOS. 11 AND 12 OF DRIFT BARRIER.

FIG. 11.—FIRST UNIT, LOOKING WEST, DURING FLOOD OF DECEMBER, 1915.

PLATE LXX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXX, No. 1377.
WOLFF ON
DESIGN OF A DRIFT BARRIER.



left in the block, loop twice around the pipes, and pass back through another set of holes, beyond which the ends are fastened to the line with wire-rope clips. The end of the tunnel is sealed with a 10-ft. block of concrete with holes through which the cables pass. The intermediate portion is timbered, with 3 ft. between the posts. A gallery extends the full length of the tunnel above the cables and blocks, for inspection and renewal of cables.

The other anchorage is a block of concrete of 480 cu. yd., in which are embedded nine 2-in. eye-bars, 20 ft. long, to which the cables are attached. The arc of the cables is maintained through three triangles made of built I-beams which are anchored down stream to small concrete blocks. To the vertical face, which is up stream, the cables are fastened with U-bolts. They are supported by slabs of reinforced concrete, 1 ft. thick, with aprons in all directions on a 1 in 3 slope, but are not anchored to these. The intermediate spacers are 3 by 5-in. angles.

Two jam piers are set a short distance up stream, in order to break up any large masses of drift.

A small auxiliary pier with an anchored I-beam triangle was placed in the main channel to absorb the first impact of drift in the first flood and prevent excessive distortion of the line of cables.

After the fall flood period, the extension of the barrier across the valley was begun. The design adopted was based on the same general idea as the first unit, but was much modified in detail.

On the assumed data, as stated previously, the horizontal pressure to be taken up by the anchorages is 1 ton per lin. ft. This is irrespective of the length of span. Assuming a coefficient of friction of 0.5, the weight of concrete at 2 tons per cu. yd., and a safety factor of 2, it is evident that 3 488 cu. yd. are necessary for anchorages in the 1 744 lin. ft. of the extension, whether the spans are long or short. The size of the piers is proportionate to the length of the span. In order to make the spans self-supporting, and thereby eliminate intermediate supports, it was decided to make them as short as possible, within the limits of leaving ample waterway and of having sufficient yardage in each pier so that, with the height necessarily fixed by the number of cables, the design of the pier could be such as to keep the center of gravity low. A distance of 64 ft. from center to center of piers was adopted. Ten horizontal cables were used, and two for

diagonal trussing. The horizontal cables were given a sag of 1 ft. in the center to allow a deflection of that much (or approximately 1 in 40) for the clear span. They are of 6 by 19 crucible-steel wire rope, 1 in. in diameter, and have an ultimate strength of 30 tons each. They are fastened with wire-rope clips to eye-bars of 1½-in. mild steel extending through the middle of the pier. The piers are diamond-shaped, and have a center height of 11 ft.; their tops have a slope of 1 in 3 in order to allow drift to ride up on them, and the sides are battered 4 in. to the foot. The details of the design are shown on Plate LXX.

The axial grade of the river at this point is 0.7%, which gives a grade of 0.4% along the line of the barrier.

The piers rest on a hard gravel and boulder foundation, evidently old river bottom, and are carried from 3 to 5 ft. below low-water grade.

It was not considered necessary to go below the scour line, and it is expected that the piers will settle out of line and level after the drift jam has begun to form. A system of small ditches will conduct the water away from the barrier across the made flat to the channel, leaving the river to cut the remainder. Figs. 4 to 9 show various features of the barrier, and Figs. 10 and 11 show some of the drift collected by it during the floods of November and December, 1915.

The underlying idea of the whole design is eventually to form across the valley a log jam which will in effect be a dam, thereby spreading the stream over a width of from 1 800 to 2 000 ft., instead of 400 ft. as heretofore, diminishing both the depth and the velocity and causing drift to be deposited, or to become interlaced and held, instead of being carried over the crest, thus further strengthening the dam itself and eliminating a maintenance charge for the removal of drift. If the natural actions of the river are a criterion, the present channel will become filled, the current being forced farther and farther across the valley as successive spans become impassable, and sand and gravel will reinforce and solidify the accumulation of débris in the dead water.

DISCUSSION

Mr. Chittenden.

H. M. CHITTENDEN,* M. AM. SOC. C. E. (by letter).—The writer believes that engineers will be interested in a brief description of the flood problem to which the drift barrier described by the author pertains. Those who have visited the Puget Sound country will recall that the overland route between Seattle and Tacoma is through a flat open valley behind a range of hills which rise 300 or 400 ft. above the level of the plain. This valley was once an arm of Puget Sound, but it has been filled up by the detritus brought down from the Cascades. Four principal streams have been the agencies in this filling process. Two of these—Cedar and Green Rivers—have always discharged to the northward through the Duwamish River into Elliot Bay, the harbor of Seattle. One, the Puyallup, has always discharged into Commencement Bay, the harbor of Tacoma. The fourth, the White River, largest of all and by far the heaviest detritus carrier, lies between the Green and the Puyallup, and drains the northern slopes of Mount Rainier. It has built up a detritus cone across the valley previously referred to, and this cone has reached the considerable elevation of 100 ft. above sea level where the stream debouches from the foot-hills. On the top of this cone, during the process of its growth, White River has flowed in unstable equilibrium for indefinite ages, winding sometimes one way and sometimes the other. Since the valley has been occupied by white men, it has flowed most of the time to the north, and the name has attached to the course of the stream for a considerable distance in that direction; the connection to the south with the Puyallup bears the unromantic name of Stuck River. The boundary between King County, in which is the City of Seattle, and Pierce County, in which is the City of Tacoma, runs very nearly along the summit of the detritus cone. When White River has flowed north, King County has had to take care of it, with its destructive freshets, and when it has flowed south, the same burden has been thrust upon Pierce County. This situation has developed controversy between the counties which has occasionally led to litigation and sometimes to something worse.

This was the situation when the flood of November, 1906, which seems to have been the greatest since the valley was occupied by settlers, carried White River bodily over to the Puyallup and carved so deep a channel past the point of divergence as to indicate that the river would remain there for a long time. King County thought it would be good policy to make this action of Nature permanent, and commenced building a dam which should prevent the river from ever flowing north again; but, after a little work had been done, the

* Seattle, Wash.

Mr. Chittenden.

plan was abandoned, probably because of protests from Pierce County. The writer was at the time in charge of the Federal Engineer District of Washington, with headquarters at Seattle, and was appealed to for advice and assistance in solving the difficult problem which had arisen. As no funds were available from any public source, he suggested local contributions sufficient to make a survey and report, and, in compliance with this suggestion, the sum of $4 000 was raised by the two counties and the four railroads operating in the valley. A voluntary board was organized consisting of the two county engineers, the four resident railway engineers, two U. S. assistant engineers, and the writer. A comprehensive survey was made, and the report of the Board's findings was duly made public.

The report recommended that White River be permanently held to the south slope, but that King County pay the major share of the cost of maintaining it there. The recommendation met with popular approval, but it was a long time before the Commissioners of King County could be swung into line, and then only as a compromise of litigation started by Pierce County. An agreement was finally reached, however, and the work is now being carried out substantially in accordance with the recommendations of the Board. These embraced a general rectification of the Puyallup River and an enlargement of the Stuck, the whole work to be fortified by an elaborate system of bank protection and levees. In order to relieve the stream of the menace which comes from the enormous quantities of drift carried in flood time, the Board recommended the construction of a drift barrier on White River, and it is this structure which is the subject of the paper.

The method of functioning of this barrier is well described in the last paragraph of the paper. The writer will not discuss the technical details of the structure further than to say that they seem to be well adapted to their purpose. The Board contemplated a pile structure, but this would have proved impracticable because of the great depth of very heavy gravel, which would have made it impossible to secure the necessary penetration. The writer's only criticism of the work as constructed relates to its location. The Board chose a location at the narrowest part of the valley below the source of nearly all the drift. For some reason, of which the writer has never seen a satisfactory explanation, the location was moved 3 miles up stream to a site which, from a construction point of view, was certainly not superior to that selected by the Board, and has the serious defect of leaving about 3 miles of prolific drift-producing territory below it.

Figs. 12 and 13 show the White River and the driftwood brought down by the 1906 flood.

FIG. 12.—DRIFTWOOD BROUGHT DOWN BY WHITE RIVER IN THE 1906 FLOOD.

FIG. 13.—DRIFTWOOD BROUGHT DOWN BY WHITE RIVER IN THE 1906 FLOOD.

FIG. 14.—SAME VIEW AS FIG. 12, AFTER CLEARING RIVER CHANNEL 1 MILE BELOW DIVERSION DAM.

FIG. 15.—SAME VIEW AS FIG. 13, AFTER CLEARING RIVER CHANNEL 2 MILES BELOW DIVERSION DAM.

FIG. 16.—2 040 CORDS OF DRIFT COLLECTED WITHIN A RADIUS OF 800 FEET, AT POINT SHOWN IN FIG. 12, ABOUT 1 MILE BELOW DIVERSION DAM.

FIG. 17.—DRIFT BARRIER SHOWING DRIFT ACCUMULATION, MARCH, 1916.

FIG. 18.—DRIFT BARRIER, MARCH, 1916.

W. J. ROBERTS,* M. AM. SOC. C. E. (by letter).—The author has given such an excellent description of this unique structure, the Drift Barrier, that little of value can be added. Mr. Roberts.

H. M. Chittenden, M. Am. Soc. C. E., has also given a clear description of the flood problem to which the Drift Barrier pertains.

The funds for improving the White, Stuck, and Puyallup Rivers are provided by a joint agreement between King and Pierce Counties, Washington, 60% being paid by King and 40% by Pierce County. A sum of $250 000 is available each year for 6 years, and a maintenance fund of $50 000 a year for 99 years is provided.

The improvement was begun in January, 1914, and the largest expenditure to date has been for the rectification of the river.

Closely following the dredge, the banks of the new channels are protected from erosion by concrete paving on the slopes and a brush mattress at the toe, ballasted with rock.

The rainy season of 1915-16 brought four high waters, one of which was higher than any since the flood of November, 1906. The Barrier was tested severely by these floods, and demonstrated fully its value and efficacy.

Its location has been questioned by Gen. Chittenden. At the site chosen by the Chittenden Board in 1907 a diversion dam has since been constructed to prevent further flow of White River northward. The present site was chosen for the principal reason that it was feared such a structure, at the site originally selected, would jeopardize the diversion dam. The criticism that changing the site to a point 3 miles up stream would leave drift-producing territory below it, is a natural one. It should be noticed, however, that Figs. 12 and 13, are reproductions from photographs of drift-producing territory below the original proposed site for the Drift Barrier. This territory, as well as the entire channel from the Barrier to the Sound, must be thoroughly cleared before the improvement is completed. The removal of these "wrack heaps" is expensive, but is fundamental in rectifying a stream of this character.

* Tacoma, Wash.

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ELECTRICAL

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657	7.65	39	Cost of Construction, Operation, and Valuation of Steam-Electric Power Plants (1912 Partial).
696	2.95	30	Extraction of Nitrogenous Product from the Atmosphere (1913 Partial).
730	2.60	15	References on Reinforced Concrete Poles in the Official Gazette of the United States Patent Office (1913 Partial).
782	2.60	20	Submarine Cable Laying (1914 Partial).
799	8.25	10	Method of Field Observation to Determine the Physical Condition of Pole Lines (1914 Partial).
800	8.45	15	Pole Lines: Relation Between Physical Condition and Value (1914 Partial).
821	4.20	3	Average Cost of Investment in Telephone Plant per Telephone (1914 Partial).
902	5.50	27	Chlorine and Liquid Chlorine Production and Cost (1915 Partial).

MARINE

22	0.35	5	Dimensions of Dry Docks on the Atlantic Coast Large Enough for Trans-Atlantic Steamers (1900 Partial).
49	0.00	1	Loading Cattle on Steamboats (1901 Partial).
157	3.50	33	Japanese Shipbuilding and Docks (1905 Partial).
289 289a	22.00	331	Dry Docks (1912 Partial).
361	1.25	17	Screw Propellers (1908 Partial).
429	2.25	18	Brooklyn Dry Docks (1909 Partial).
461	5.00	11	Concrete Barges (1909 Partial).
466	3.00	2	Early Foreign Inventors of the Steamboat (1910 Partial).
578	14.65	85	Speed and Resistance of Vessels in Shallow Channels (1911 Partial).
732	4.55	5	Dredges Built for the United States Government by the Ellicott Machine Company (1913 Partial).
751	6.00	15	Sliding and Floating Gates or Caissons for Dry Docks or Locks (1914 Partial).
790	2.80	1	Gas Producer Tugs for the Lehigh Valley Coal and Navigation Co. (1914 Partial).
885	6.50	63	Equipment for Handling Cargo at Waterway Terminals (1915 Partial).
945	16.20	68	Shipbuilding and Repair Plants (1916 Partial).
952	4.90	23	Shipbuilding Berth Equipment (1916 Partial).
960	12.10	22	Marine Railways for Docking Ships (1916 Partial).
973	7.60	16	Self-Propelled Steel Barges (1916 Partial).

MECHANICAL

14	1.00	15	Coal Handling Machinery (1901 Partial).
24	1.40	5	Holding Power of Drift-Bolts (1901 Partial).
25	0.90	14	Passenger and Freight Elevators (1902 Partial).
29 29a	12.10	135	Manufacture of Calcium Carbide and Acetylene (1907 Partial).
30	0.70	2	Mond Process of Gas Production (1901 Partial).
33	0.35	15	List of Iron and Steel Works in Germany (1901 Partial).
50	0.35	6	Smoke Prevention (1902 Partial).
63	1.14	6	Tower Derricks (1903 Partial).
74	0.50	3	Tests of Hooks and Shackles (1904 Partial).
101	3.50	7	Central Control of Valves in Steel Works (1904 Partial).
109	2.50	11	Coal Storage Plants (1904 Partial).
128	1.50	18	Kilns (1905 Partial).
142	3.25	24	Centrifugal Pumps, January, 1900, to April, 1905, Inclusive (1905 Partial).
174	1.50	11	Tramways for Lumbering (1906 Partial).
181	2.50	54	Sand-Lime Process of Making Brick (1906 Partial).
186	2.00	12	Central Refrigerating Installations (1906 Partial).

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MECHANICAL.—(Continued)

No.	Cost.*	No. of Ref.	
191	$3.75	33	Coal Tipples (1906 Partial).
212	2.15	6	Friction Losses of Liquids Other Than Clean Water (1906 Partial).
223	1.25	7	Jib Cranes (1906 Partial).
224	4.15	8	Holding Power of Anchor Bolts in Rock (1906 Partial).
229	2.70	19	Operation of Steam Shovels (1906 Partial).
287	1.00	16	Use of Cooling Towers in Connection with Steam Condensing Plants (1907 Partial).
302	4.75	22	Manufacture of Illuminating Gas in Boston and New York (1907 Partial).
314	4.00	24	Distilling Water for Use in Boilers (1907 Partial).
332	0.75	3	Koppers Coke Oven (1907 Partial).
378	4.30	32	Reinforced Concrete Coaling Stations and Coal Storage Bins, 1900-07 (1908 Partial).
387	3.50	15	Cost of Operating Modern Cold Storage Buildings and Plants (1908 Partial).
406	4.85	17	Pneumatic Conveyors (1909 Partial).
536	9.95	55	Briquetting and Briquetting Presses (1911 Partial).
580	21.15	170	Mechanical Handling of Material (1911 Partial).
608	1.00	4	A Traveling Bridge Suspended from a Cableway for Making Fills (1912 Partial).
614	4.25	9	Cargo Handling Devices on Unprotected Coast Lines (1912 Partial).
627	1.00	8	Glass Manufacture (1912 Partial).
641	9.60	48	Motor Trucks, Gasoline and Electric, Cost of Operation (1912 Partial).
678	6.00	21	Prevention of the Breakage of Coal in Handling (1913 Partial).
702	14.30	42	Cost of Gas Pipe and Cost of Laying It (1913 Complete.)
710	7.00	71	American Foundry Plants (1913 Partial).
749	2.10	21	Manufacture of Rubber Tires (1914 Partial).
753	7.50	14	Suction Conveyors (1914 Partial).
763	2.50	12	Chemical or Physical Reaction Involved in Soldering by the Use of Zinc Chloride Flux (1914 Partial).
785	3.30	24	Manufacture of Mineral Oils and Asphalts (1914 Partial).
786	3.90	45	Lubrication and Lubricating Oils (1914 Partial).
794	4.20	9	Sand Blast for Cleaning Bridges and Buildings (1914 Partial).
802	3.25	16	Rate-Setting for Machine-Shop Work (1914 Partial).
803	7.80	18	Location of Cross-Compound Horizontal and Vertical Triple Pumping Engines, Capacity 12 000 000 to 20 000 000 Gallons (1914 Partial).
823	2.30	6	Press Board or Presspahn (1914 Partial).
863	4.50	2	Pumping Machinery for Molasses (1915 Partial).
871	10.10	40	Aerial Tramways for Passenger Service to Great Heights on Heavy Grades (1915 Partial).
883	2.10	2	Cost of Erection and Equipment of Sugar Refining Plants (1915 Partial).
884	2.10	4	Cost of Erection and Equipment of Cold-Storage Plants (1915 Partial).
886	9.10	74	The Use of Coal Dust for Making Steam in Locomotive and Stationary Engines (1915 Partial).
906	8.45	25	Definition of "Coking", on Petroleum Coking, and the Derivation of the Word Coking (1915 Partial).
931	5.40	15	The Use of Natural Gas for Production of Power (1916 Partial).
946	10.90	67	Cranes, 30 Tons Capacity or Greater (1916 Partial).
948	14.30	67	Machine Shops of Large Capacity (1916 Partial).
949	14.80	68	Foundries and Forge Shops of Large Capacity (1916 Partial).
951	1.80	6	Boiler Shops (1916 Partial).
987	10.20	13	Explosions Caused by Illuminating Gas Transmitted Through Mains in the Street (1916 Partial).

METALLURGICAL

75	1.00	16	Smelting and Ferro-Titanium. Articles by A. J. Rossi (1904 Partial).
120	1.25	14	Furnaces, Furnace Plants and Rolling Mills (Books) (1904 Partial).
218	7.25	56	Bibliography on the Nodulising and Desulphurisation of Fine Ores and Pyrite-Cinders (1906 Partial).
272 272a	12.00	150	Reduction of Bauxite and the Manufacture of Aluminum and Aluminum Products (1907 Partial).

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METALLURGICAL—(Continued)

No.	Cost.*	No. of Ref.	
492	$3.65	5	Relative Heat Required to Reduce Calcite and Dolomite Stone in a Blast Furnace (1910 Partial).
493	7.00	30	Relative Efficiency of Beehive and By-Product Coke in the Operation of a Blast Furnace (1910 Partial).
767	2.00	7	Chrome Brick (1914 Partial).
768	3.10	15	Fire Brick (1914 Partial).
882	2.40	17	Nodulization or Sintering of Flue Dust and Iron Ores (1915 Partial).
947	16.40	90	Steel Works of Large Capacity (1916 Partial).
953	2.40	15	Plants for Manufacture of Structural Steel Members (1916 Partial).

MILITARY

969	18.90	51	Layout and Design of Gun and Projectile Plants (1916 Partial).
975	18.90	64	Armor Plate (1916 Partial).

MINING

55	4.05	3	Poetsch Freezing Process (1902 Partial).
126	0.50	1	Annual Production and Value of Phosphate Rock in the United States from 1890–1903 (1905 Partial).
146	3.00	29	Dredging of Auriferous Sand and Gravel (1905 Partial).
159	1.50	8	Kind and Chaudron Method of Shaft Sinking Through Water-Bearing Solids (1905 Partial).
283	2.35	7	Stone or Ore Crushing Machinery, Crushing Pieces 15 In. or Larger (1907 Partial).
435	0.90	8	Development of Oil Fields in Texas (1909 Partial).
436	0.45	3	Development of the Anthracite Coal Fields of Arkansas (1909 Partial).
469 469a	4.65	40	Concrete-Lined Shafts (1910 Partial).
496	5.50	26	Mining by Means of Winzes or Glory Holes (1910 Partial).
545	3.10	21	Tilly Foster Iron Mine, and Mineral Deposits in Its Immediate Vicinity (1911 Partial).
574	5.20	37	Methods of Lining Circular and Elliptical Shafts (1911 Partial).
576	4.55	7	Earthern Reservoirs for the Storage of Crude Petroleum (1911 Partial).
597	3.60	3	The Effect of Sea Water on Dynamite (1911 Partial).
605 605a	10.45	78	Rock Excavation (1914 Partial).
807	4.20	15	Sinking Shafts More Than 1 500 Ft. Deep in Rock (1914 Partial).
817	1.80	4	Methods of Locating Oil (1914 Partial).

MISCELLANEOUS

20	0.35	5	Dictionaries Containing Technical and Engineering Terms (1903 Partial).
26	0.00	51	List of Books on Civil Engineering for a Small Library (1901 Partial).
27	0.70	28	List of American Publications on Water Supply of Towns, Irrigation, Hydraulic Machinery, Practical Bridge Designing, Practical Tunneling, Railway Signaling, Railway Permanent Way Material and Construction, Road and Street Design and Construction, and Electric Traction (1902 Partial).
39	1.00	6	Pipe Lines and Tanks for Oil Supplies (1901 Partial).
83	0.50	5	Civil Service (1904 Partial).
87	3.50	49	Geologic History of Manhattan Island and Long Island (1904 Partial).
122	2.75	42	Forest Preservation (1904 Partial).
135a	8.15	13	"Gumbo", Definition of. (Includes Search No. 135) (1913 Partial).
150	0.00	26	Selected List of Engineering Indexes (1905 Partial).
152	4.50	16	Meaning of the Terms "Net Section" and "Neat Section" (1905 Partial).
155	0.50	1	Method of Finding Center of Population (1905 Partial).
164	2.50	24	Ethics for Civil Engineers (1905 Partial).
173 173a	27.40	32	Character and Excavation of Hardpan (1911 Partial).

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MISCELLANEOUS—(Continued)

No.	Cost.*	No. of Ref.	
197	$5.00	99	List of Technical Books for a General Library to Cost About $250 (1905 Partial).
233	0.80	12	Do the Field Notes of an Engineer Belong to the Employer or the Employee? (1906 Partial).
266	6.75	47	Peat: Its Use as a Fuel, etc.; Location of Principal Peat Beds (1907 Partial).
307	24.10	62	Cost Allowances in Per Cent. Made for Engineering, Contractor's Profit, and for Expense for Organizing Corporations, in Estimates for Public Works (1907 Partial).
340	6.20	29	Preparation and Manufacture of Coal-Tar and Asphalt (1908 Complete).
349	0.90	6	Cost Keeping and Methods of Estimating Cost (1908 Partial).
350	6.55	36	Details of the Cost of Construction (1908 Partial).
365 365*a* 365*b*	11.70	63	Hydraulic Excavation (1914 Partial).
372	10.70	48	Ozone and Its Uses (1908 Partial).
403	3.85	10	Rate Per Cent. at Which an Annual Income Should be Capitalized to Determine the Value of Appraisal of Award (1909 Partial).
404	1.10	8	Fish Glue (1909 Partial).
407	15.75	277	Cotton Industry (1909 Complete).
424	4.30	40	Bonus, Premium, and Piece Work Systems of Remunerating Labor (1909 Partial).
441	1.35	5	Oil Refineries in Texas (1909 Complete).
462	0.50	10	Decimal Filing Systems (1909 Partial).
468	4.00	37	Engineering Office Records (1910 Partial).
489	0.50	1	Six-Place Table of Trignometric Functions (1910 Partial).
497	4.75	40	Oil Transportation (1910 Partial).
519	5.15	25	Early German Engineers in the United States (1910 Complete).
530	0.80	9	Natural Gas in Louisiana, Ohio, and Other Parts of the United States (1910 Partial).
587 587*a*	19.60	158	Legislation Regulating the Practice of Engineering in the United States (1911 Partial).
594	†324.50	1236	Valuation of Public Utilities (1912–13 Partial).
594*b*	260.00	947	Valuation of Public Utilities (1915 Partial).
628	1.30	3	The Integraph (1912 Partial).
645	3.25	8	Expert Evidence in Engineering Legal Cases (1912 Partial).
649	1.30	7	Hydraulic Sluicing on the Canadian Pacific Railway and at Seattle, Washington (1912 Partial).
677	8.15	43	Turpentine (1913 Partial).
680	7.15	11	Extra Compensation Allowed for Delay in Completion of Contract (1913 Partial).
682	3.90	14	Articles and Discussions by Mr. Horace E. Horton, published in the *Journal* of the Western Society of Engineers (1913 Complete).
683	3.25	19	City Engineers or Officials of Similar Position (1913 Partial).
742	6.30	3	Ownership of Engineers' Drawings (1913 Partial).
754	6.20	21	Overhead Bridge, Viaduct, Subway and Tunnel, Definition (1914 Partial).
759	21.65	238	List of Professors or Instructors in Engineering, Elected Members, Associate Members, and Associates of the American Society of Civil Engineers (1914 Partial).
787	3.60	66	List of Corporations Employing Large Numbers of Engineers (1914 Partial).
813	13.10	246	List of Technical Societies.
832 832*a*	11.00	28	The Engineer as a Citizen (1915 Partial).
839	1.10	7	Regulations of Engineering Departments as to Reports and Drawings (1915 Partial).
854	5.00	9	Value of Land Sold for the Use of Public Utilities (1915 Partial).
856	1.30	19	List of Universities in Canada, Central and South America (1915 Partial).
857	7.50	121	List of Technical Societies in North and South America (1915 Partial).
860	3.10	20	Methods and Costs of Harvesting and Storing Natural Ice (1915 Partial).

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† This bibliography has been published in *Transactions*, Am. Soc. C. E., as Paper No. 1280, and can be obtained for 50 cents.

MISCELLANEOUS—(Continued)

No.	Cost.*	No. of Ref.	
925	$7.20	40	Engineering Education (1916 Partial).
926	2.30	10	English for the Engineer (1916 Partial).
927	0.50	3	Public Recognition of Engineers (1916 Partial).
930	3.90	17	Food for Armies and Workers in Construction Camps (1916 Partial).
941	21.30	87	The Industrial Use of Oxygen (1916 Partial).
943	6.90	65	List of Engineering Works for a General Library (1916 Partial).
955	11.70	72	Quotations (Ex. Dock) New York for Sulphite, Bleached, Unbleached; Sulphate, Bleached, Unbleached; Kraft Pulp; as given in *The Paper Trade Journal* for the Years 1909–1914 Inclusive (1916 Partial).
976	1.10	13	Principles of Industrial Organization (1916 Partial).

MUNICIPAL

84	1.50	28	Municipal Ownership (1904 Partial).
119	1.00	1	House Numbering Systems (1904 Partial).
851	8.00	78	City Planning (1915 Partial).
908	5.20	64	City Government by City Manager (1915 Partial).
986	3.90	7	How to Apportion Cost of City Improvements, Especially Street Openings (1916 Partial).

RAILROADS

8	2.50	19	Ventilation and Construction of Tunnels (1903 Partial).
15	0.75	6	Grade Crossings (1903 Partial).
28	0.70	4	Cost of Transferring Freight Cars by Ferry (1902 Partial).
41	0.75	7	Narrow-Gauge Railroads (1903 Partial).
42	1.00	22	Ship Railways (1900 Partial).
43	1.00	6	Resistance on Railroad Curves (Formulas) (1901 Partial).
44	3.50	21	Train Resistance (1901 Complete).
52	0.35	6	Railroad Signaling and Interlocking (1902 Partial).
78	3.00	30	Reinforced Concrete in Railway Work (1904 Partial).
97	3.75	78	Railroad Terminals (1904 Partial).
106	3.25	27	Taxation and Valuation of Railways (1904 Partial).
111	2.25	18	Maximum Curves and Grades (1904 Partial).
115	4.50	12	Tonnage Rating of Locomotives as Affected by the Length of Ruling Grades (1904 Partial).
118	1.75	5	The Barschall Rail Joint (1904 Partial).
140	1.75	8	Systems of Maintenance of Way Bookkeeping in Use on Railroads (1905 Partial).
144	3.00	13	Freight Traffic in New York City (1905 Partial).
162 162a	13.15	102	The English Channel Tunnel (1914 Partial).
182	1.25	6	Prices of Steel Rails (1906 Partial).
188	3.50	39	Reinforced Concrete Ties (1906 Partial).
201	1.25	6	Sand Track (Derailing Device) (1906 Partial).
211	18.00	242	Selected Bibliography on Rails (1906 Partial).
225	7.00	82	Ballast (1906 Partial).
253	3.00	19	Railroad Embankments Carried Over Swamps (1907 Partial).
256	7.65	27	Slips of Earthwork (1907 Partial).
258	4.25	98	Construction of Long Railroad Tunnels (1907 Partial).
260	7.00	74	Motor Cars, Electric, Gasoline, Compressed Air (1907 Partial).
262	7.15	106	Rack Railroads (1907 Partial).
277	2.00	14	New York Central Type of Under-Contact Third Rail (1907 Partial).
279	2.60	24	Cross-Sections of Single-Track Railway Tunnels in the United States (1907 Partial).
285 285a	8.85	58	Intensity of Pressure Between Locomotive and Car Wheels and Rails (1915 Partial).
290	7.05	44	Passenger and Freight Cars Used in Tropical Countries on Broad- or Narrow-Gauge Railroads, 1898–1907 (1907 Partial).
292	10.00	7	Time for Leaving Arch Centers in Place to Support Concrete Lining for Railroad Tunnels (1907 Partial).
304	4.95	79	Tunnel Construction (1907 Partial).
313	0.50	1	Prior Location and Occupation of Right of Way (1907 Partial).
320	5.85	84	Construction of the Mont Cenis, St. Gothard, Arlberg, and Simplon Tunnels (1907 Partial).

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RAILROADS—(Continued)

No.	Cost.*	No. of Ref.	
327	$4.30	4	Cost of Driving Subaqueous Tunnels, 9 Ft., or Greater, in Diameter and from 1 000 to 2 000 Ft. in Length, by Means of Compressed Air (1907 Partial).
333 333a 333b	48.60	362	Track Elevation and Depression in Cities (1908 Partial).
336	0.75	1	Average Speed with Which Freight is Handled (1908 Partial).
337	5.00	31	Hungarian Government System of Railway Rates (1908 Complete).
344	0.50	1	The Nationalization of Swiss Railways (1908 Partial).
347	3.40	7	Limit Usually Assigned for Batter of Retaining Walls (1908 Partial).
351	5.65	52	Economics of Curve and Grade Reduction (1908 Partial).
357	7.00	28	Marginal Railroad in West Street, New York City (1908 Partial).
364	2.00	6	Density of Population in Relation to Earnings of Interurban Railways (1908 Partial).
367	3.75	17	Crossing of Steam Railroads at Grade and the Regulations Governing the Operation of Trains at Such Points (1908 Partial).
368	1.05	9	Ogden-Lucin Cut-Off (1908 Partial).
370	4.50	34	Car Ferries, including Transfer Bridges, Inclines, Cradles, or Elevators and Other Accessories (1908 Partial).
373	4.05	24	Intercontinental Railway (Pan-American) (1908 Partial).
374	17.35	216	Narrow *versus* Broad Gauges for Railroads (1908 Complete).
379	4.95	75	Driving Tunnels in Earth (1908 Partial).
392	0.50	7	Pennsylvania Railroad Tunnels under the Hudson River, and Terminal in New York City (1908 Partial).
397	0.90	12	Detroit River Tunnel (1908 Partial).
398	1.15	19	Monorail Railways (1908 Partial).
399	2.05	27	Catenary Construction (1908 Partial).
400	1.50	10	Data Relating to Accidents to Passengers and Employees on Railroads (1908 Partial).
409	0.90	2	Pressures on Culverts in High Embankments (1909 Partial).
444	5.40	45	Reinforced Concrete Retaining Walls (1909 Partial).
445	3.40	4	Cost of Operating Railroad Yards (1909 Partial).
452	2.70	1	Quotations on Prices for Angle-Bars and Track Bolts with Square Nuts (Mill Prices) (1909 Complete).
467 467a	11.45	84	Rail Specifications (1912 Partial).
475	4.25	9	The Relation of the Reduction of Grades and Curves, etc., in Railroads to Operating Expenses (1910 Partial).
476	1.75	22	Electrification of Steam Railroads, Advantages and Disadvantages (1910 Partial).
490	9.65	41	Strength and Wear upon Large Cables (1910 Partial).
498 498a 498b 498c	37.90	384	Subaqueous Tunnels (1913 Partial).
508	2.25	7	Economic Result of Double-Tracking Railroads (1910 Partial).
520	15.00	176	Electrification of Steam Railways (1910 Partial).
529	5.35	18	Tunnel-Boring Machines for Tunnels in Rock (1910 Partial).
546	1.00	7	Weight of Rail Used on the Main Lines of the New York Central and Pennsylvania Railroads (1911 Partial).
547	4.55	16	Sand Patch Tunnel, Pittsburgh and Connellsville Railroad (1911 Partial).
549	3.10	15	Railroads in Cuba (1911 Partial).
551	7.15	83	Problems Arising from Grade Crossings and Their Elimination (1911 Partial).
554	4.90	6	Certain Articles on the Simplon Tunnel (1911 Complete).
559	5.55	31	Organization for, and Actual Physical Valuation of, Railroads by State Railroad Commissioners (1911 Partial).
560	10.10	46	Size of Cross-Ties in Relation to the Stresses to Which They are Subjected (1911 Partial).
591	1.30	1	Romopac Rails (1911 Partial).
601	3.90	28	Inclined Planes for Railways (1912 Partial).
610	14.65	18	Clearance Line of Bridges in Railroad Terminals (1912 Partial).
611	18.40	130	Arrangement of Tracks at Passenger Terminals (1912 Partial).
613	3.25	3	Details of Rack Rails, etc., Ferro Carril Dominicana (1912 Partial).
623	3.25	2	Special Rates on Railroads for Material Transported for the Reclamation Service (1912 Partial).

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RAILROADS—(Continued)

No.	Cost.*	No. of Ref.	
637	$11.85	41	The Economics of Railway Track Labor (1912 Partial).
646	2.80	9	Woods Used for Railway Ties in America: Their Cost, Life, and Locality of Supply (1912 Partial).
647	4.10	38	Treated *vs.* Untreated Railway Ties; Cost of Treating, Results, and Location of Creosoting Plant (1912 Partial).
651	3.90	17	Car Transfer Inclines, Cradles, and Aprons (1912 Partial).
667	1.30	9	Economical Rating of Freight Trains (1913 Partial).
675	3.60	14	Construction of the Loetschberg Tunnel (1913 Partial).
676	1.80	10	Railway Engineering Organization (1913 Partial).
687 687a 687b	23.75	37	Reinforced Concrete Retaining Walls, L or Inverted T-Form, Constructed Prior to 1899 (1916 Partial).
689	3.90	21	History of the West Shore Railroad from Its Inception to the Time of Its Completion, 1871 to 1883 (1913 Partial).
690	10.10	82	Suspended Monorail Tramways (1913 Partial).
701	4.25	28	Organization for Maintenance and Supervision of Track (1913 Partial).
703	5.55	5	Driving Headings in Tunnels, Monthly Progress Exceeding 683 Ft. (1913 Partial).
705	7.15	26	Umbrella Train Sheds (1913 Partial).
709	13.00	63	Tonnage Rating (1913 Partial).
712	2.30	21	Car and Locomotive Shops for Railroads (1913 Partial).
718	2.90	9	The Railway Station at Basle, Switzerland (1913 Partial).
719	9.75	59	Hauling Materials for Grading in Connection with Railroads (1913 Partial).
720	7.15	30	Classification of Material in Railway Construction (1913 Partial).
726	4.90	13	Statistics of Accidents on Railroads at Grade Crossings (1913 Partial).
727 727a	7.50	13	Overbreak in Solid Rock Cuts (1914 Partial).
728	4.40	18	Grade Crossing Elimination (1913 Partial).
731	7.50	33	Handling Baggage, Mail, and Express at Passenger Terminals (1913 Partial).
735	1.00	1	Railway Interlocking Plants: The Year They were Introduced in this Country (1913 Partial).
738	2.60	33	Tunnels in Earth (1913 Partial).
739	11.70	129	Electrification of Steam Railroads (1913 Partial).
750 750a	5.70	39	Lining Railroad Tunnels with Concrete (1914 Partial).
774	2.30	5	Law of the State of New York in Regard to Calculation of Earthwork (1914 Partial).
777	5.20	39	Marginal Freight Railways for Harbor Terminals (1914 Partial).
779	3.90	14	Damages Awarded for One Railroad Crossing Another at Grade (1914 Partial).
780	1.80	8	History of Standard Railroad Construction Contract (1914 Partial).
788	4.30	8	Widening of Single-Track Tunnels without the Interruption of Traffic (1914 Partial).
797	1.30	12	Increase of Mileage of Railroads in Missouri, Oklahoma, Kansas, and Texas, 1900–11 (1914 Partial).
805	115.70	820	The Development of Iron and Steel Rails (1914 Partial).
806	5.00	3	Waste in Solid Rock Cuts (1914 Partial).
812	7.00	5	Reinforced Concrete Retaining Walls Without Expansion Joints, More than 50 Ft. Long (1914 Partial).
835	1.30	8	Spark Arresters (1915 Partial).
837	1.30	3	Metal and Concrete Ties (1915 Partial).
845	5.00	30	Creeping of Rails and Anti-Rail Creepers or Rail Anchors (1915 Partial).
864	1.30	2	Road Opened at Grade, Crossing the Main Line of a Railroad (1915 Partial).
865	2.10	7	Division of Values of Railroad Property Between Freight and Passenger Service (1915 Partial).
878	6.00	13	Comparison of Electric and Steam Power for Heavy Freight Trains on Mountain Railroad (1915 Partial).
881	3.40	41	Railroad Terminal Equipment (1915 Partial).
889	5.00	14	Cost and Area of Passenger Station Buildings, in Cities of 150 000 to 400 000 Population (1915 Partial).
913	1.70	6	The Spacing of Wooden Railroad Ties (1915 Partial).
916	7.80	12	Use of Clay Blanket or other Artificial Covering for Subaqueous Tunnel Work, Prior to 1902 (1915 Partial).

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RAILROADS—(Continued)

No.	Cost.*	No. of Ref.	
918	$1.60	8	Driving a Tunnel Through Earth to be a Twin Tunnel in a Single Excavation (1916 Partial).
971	3.90	6	Character of Site of the Hudson and Manhattan Tubes from Jersey City to Cortlandt Street (1916 Partial).
977	4.90	8	Cost of Changing Railroad Gauge (1916 Partial).

RAILROADS, STREET

45	1.40	10	Inclined Planes in Use on Street Railways (1901 Complete).
110	1.50	17	Details and Design of Structural Steel in Elevated Railroads (1904 Partial).
156	3.75	8	Contract Prices of Electric Conduit Street Railways (1905 Partial).
160	2.25	8	Cross-sections of Subways, Berlin and Paris (1905 Partial).
165	4.00	7	Experimental Lines Built to Prove the Feasibility of Elevated Railroads (1905 Partial).
220 220a	12.10	62	The Use of Tee-Rails by Surface Railways, Particularly in Connection with Brick Paving (1912 Partial).
248	2.25	8	Construction of Street Railway Track, 1905-06, in Streets Paved with Vitrified Brick (1907 Partial).
249	7.50	13	Vibrations in Buildings Caused by Traffic in Tunnels or Underground Railways (1907 Complete).
342	1.90	35	Third Avenue Railway, New York City (1908 Complete).
417	1.35	10	Subaqueous Tunnel Built by the Boston Transit Commission at Boston, Mass. (1909 Partial).
447	2.60	3	Cost of Track, Pole, and Wire Work of Overhead Trolley Systems in City Streets (1909 Partial).
531	1.15	20	Names of Managers of Street Railways in Certain European Cities (1910 Complete).
532	10.40	83	Zone System Rates of Fare in Certain European Cities (1910 Partial).
553	4.55	16	Single-End Street Railway Cars (1911 Partial).
558 558a	5.20	22	Operation of Terminals on Street Railways for Single-End Cars (1911 Partial).
596	15.45	93	Subway Systems as Practical and Economic Mediums of Urban and Interurban Transportation for Passengers and Freight (1911 Partial).
664	2.60	7	The First Elevated Street Railway in America (1912 Partial).
668	6.50	28	Comparative Merits of Tee-Rail and Grooved Girder Rail for Street Railways (1913 Partial).
734 734a	6.00	17	Thickness of Railroad Ties, and Weights of Cars Used on Elevated Railroad Structures in the United States (1913 Partial).
843	5.00	29	Construction of Street Car Tracks in Paved Streets (1915 Partial).

ROADS AND PAVEMENTS

46	0.50	12	Statistics of Macadam and Other Street Paving (1901 Partial).
92	1.00	14	Paving Brick Tests (1904 Partial).
237	1.00	8	Use of Crude Petroleum on Roads (1906 Partial).
244a	7.65	116	Wood Pavements (Includes Search No. 244) (1908 Partial).
251	3.15	20	Use of Asphalt Blocks and Their Comparison with Brick Paving (1907 Partial).
261	1.25	10	Laying Out of Standard Race Tracks for Harness Horses (1907 Partial).
265	11.50	175	Street Cleaning (Covers Collection of City Refuse, but not Disposal) (1907 Complete).
267	4.25	39	Statistics of Street Paving Other than New York City, 1896 to Date (1916 Partial).
317 317a	36.70	357	The Use of Tar, Bitumen, Oil, Cement, etc., on Macadam Roads, Before and After Construction (1907 Complete).
356	6.55	69	Repair and Maintenance of Telford and Macadam Roads (1908 Partial).
383 383a	9.80	49	Different Methods of Making Assessments for Brick Pavements or other Street Improvements (1911 Partial).
418	1.95	18	Specifications for Macadam Roads (1909 Partial).
472	3.50	14	Action of Illuminating Gas on Asphalt Pavements (1910 Partial).

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ROADS AND PAVEMENTS—(Continued)

No.	Cost.*	No. of Ref.	
541	$6.20	26	Actual Tests of the Tractive Force Required to Move a Given Weight on Roads and Level Railroads (1911 Partial).
572	13.65	32	Durability of Concrete Pavement (1911 Partial).
598	5.05	5	Approximate Area of Streets and of Building Spaces in Large Cities (1911 Partial).
599	10.10	102	Highway and Permanent Pavement Design (1912 Partial).
617	4.90	21	Foundations for Highways in Soft Mud (1912 Partial).
633	8.60	9	Peekskill or Roa Hook Gravel (1912 Partial).
665	4.55	13	Automobile Race Tracks (1913 Partial).
666	7.30	39	Effect of Motor Traffic on Roads (1913 Partial).
669	6.20	51	Construction, Maintenance, and Oiling of Macadam Roads (1913 Partial).
685	14.30	16	Road Maintenance: Methods of Organization and Cost (1913 Partial).
686	1.95	4	Street Signs in Concrete Sidewalks (1913 Partial).
692 692a	117.35	375	Use of Different Kinds of Rock in the Construction of Wearing Surfaces of Roads and Pavements, and for Railroad Ballast (1913 Partial).
729	10.90	48	Municipal or State Highway Organizations (1913 Partial).
772	7.00	67	City Paving Materials (1914 Partial).
776	8.10	83	Bituminous Materials, Concrete and Reinforced Concrete as City Paving Materials (1914 Partial).
816 816a	10.20	57	Warren Brothers Company Bitulithic Pavement (1914 Partial).
820	2.40	18	Motordromes (1914 Partial).
840	5.00	53	Stone Block Pavements (1915 Partial).
842	5.00	80	Wooden Block Pavements (1915 Partial).
852	13.00	106	Financing of Highway Improvements (1915 Partial).
921	5.70	20	Amount of Friction and Pressure of Wheels on Roads of Different Kinds of Soil (1916 Partial).
938	4.00	7	Laying Out and Constructing a Horse Race Track (1916 Partial).
939	2.90	4	Pedestrian Street Traffic Census (1916 Partial).
970	2.40	6	Specifications for Length of Concrete or Artificial Stone Curbs (1916 Partial).
981	7.50	5	Asphalt over Coal or Tar Surfaces in Road Work (1916 Partial).
982	0.60	3	Dust Prevention on Roads (1916 Partial).
986	3.60	7	How to Apportion Cost of City Improvements, Especially Street Openings (1916 Partial).

SANITATION

4	1.00	12	Caisson Disease (1901 Partial).
47	1.00	8	Sewage Disposal. List of Recent Books, January 10th, 1903 (1903 Partial).
58	1.00	6	Charcoal Filter Used in Sewage Disposal (1903 Partial).
62	1.75	13	Scott-Moncrieff System of Sewage Disposal (1903 Partial).
71	0.50	7	Irrigation and Filtration Methods of Sewage Disposal (1903 Partial).
73	1.50	17	Concrete Sewers (1904 Partial).
88	3.50	18	Tests of the Strength of Vitrified Sewer Pipe (1904 Partial).
104	9.00	80	Odors and Gases from Rendering and Garbage Disposal Works (1904 Partial).
113	4.00	16	Steam Heating Plants: Capacity about 8 000 000 Cu. Ft. of Air (1904 Partial).
125 125a	4.75	34	Sewer Gauging (1916 Partial).
138	3.75	21	Relation of Run-Off to Rainfall in Sewer Districts (1905 Partial).
148 148a	9.50	125	Septic Tanks and Sewage Filtration Beds (1910 Partial).
163	2.25	6	Ducat Oxygen Process of Sewage Disposal (1905 Partial).
185 185a	5.35	52	Central Heating Stations in the United States (1908 Partial).
187a	12.15	139	Refuse Destructors (Includes No. 187) (1908 Partial).
198	6.50	50	Land Reclamation (1906 Partial).
238	4.05	19	Nuisances from Gas-Works Refuse and Treatment of the same (1906 Partial).
240	10.00	4	Methods of Securing Samples of Subaqueous Mud (1906 Partial).
241	4.50	1	Design and Construction of Cooling Tanks (1906 Partial).

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SANITATION—(Continued)

No.	Cost.*	No. of Ref.	
245	$5.85	61	Sewage Purification: Methods, Maintenance, and Results (1906 Partial).
305	5.75	20	Tests of Strength of Vitrified Pipe (Includes No. 88) (1907 Complete).
306	2.25	8	Coefficient of Friction in Vitrified Sewer Pipe (1907 Partial).
334	7.65	83	Land Reclamation by Drainage (1907 Partial).
359	5.40	19	Incinerators for the Destruction of Sludge (1908 Partial).
366	4.05	19	Durability of Small Cement Pipe (1908 Partial).
393	1.50	4	Deaths by Typhoid Fever at Pittsburgh, Pa. (1908 Partial).
419	4.95	43	Siphons for Sewerage Systems (1909 Partial).
431	1.60	9	Memphis Sewerage System (1909 Partial).
437	0.70	5	Drainage of Swamp Lands in the South (1909 Partial).
440	0.90	9	New Orleans Sewerage and Water-Works Systems (1909 Partial).
453 453a	3.90	12	Drainage of the Florida Everglades (1909 Complete).
455 455a	14.05	57	Screens Used in the Treatment of Sewage (1909 Partial).
460	5.00	7	The Absorption of Water and the Action of Frost on Vitrified Sewer Pipe (1909 Partial).
470	6.15	39	Submerged Sewer Outfalls (1909 Partial).
477	3.25	25	Grit Chambers for Sewers (1910 Partial).
481	3.65	19	Assessments for Farm Drainage (1910 Partial).
484	3.75	24	Purification of Sewage Containing Waste from Cotton and Woolen Mills (1910 Partial).
486	6.40	32	Durability and Strength of Concrete Sewers (1910 Partial).
487	3.25	11	Wear of Sewer Inverts (1910 Partial).
491	5.15	24	Sewage Disposal—Plants and Experimental Works in Germany (1910 Partial).
510	3.50	14	Amount of Infiltration of Ground-Water into Sewers (1910 Partial).
512	8.65	44	Comparative Statements of Typhoid Fever Percentages, Before and After Improvement of Water Supplies (1910 Partial).
514	4.00	24	Sewer Gauging, Dry-Weather Flow (1910 Partial).
518	6.75	28	The Amount of Bids or Cost of Sewers More Than 21 In. in Diameter Actually Built in Newark, Orange, East Orange, Montclair, Bloomfield, and Newark, or Hackensack Meadows (1910 Complete).
527	8.30	21	Use of Wooden Stave Pipe for Sewers (1910 Complete).
566	3.25	19	Methods of Sewage Disposal for Isolated Buildings (1911 Partial).
586	1.95	3	Cost of Laying Cast- or Wrought-Iron Pipe 60 In. in Diameter in Water from 50 to 75 Ft. Deep (1911 Partial).
593	2.45	1	Typhoid Fever Death Rate per 100 000 Persons during 1909, in Lawrence, Mass., and Pittsburgh, Pa. (1911 Partial).
595	3.10	29	Sewer Design and Construction (1911 Partial).
609	2.30	3	Drainage in the Yazoo Delta Between Memphis and Vicksburg, Miss. (1912 Partial).
612	20.00	37	Impurity of Street Washings (1912 Partial).
616	8.80	34	Reports on Navigation, Water Power, Sewage Disposal and Dilution, Chicago Main Drainage Canal and Sanitary District (1912 Partial).
625	18.55	64	Disposal of Night Soil (1912 Partial).
631	9.75	23	Tide Gates for Sewers not Less than 3 Ft. in Diameter (1912 Partial).
656	4.55	13	Methods of Payment for the Discharge of Sewage of one Municipality through the Sewers of an Adjacent City (1912 Partial).
659	9.60	81	Trade Waste Disposal Plants (1912 Partial).
660	7.80	26	Admission of Trade Wastes to Municipal Sewer Systems (1912 Partial).
681	3.90	12	Florida Drainage Projects (1913 Partial).
715	13.00	15	Cities in the United States and Canada where Sewerage Systems are Operated by Private Companies (1913 Partial).
716 716a	45.50	175	Opinion on Standard of Purity for Rivers and Tidal Waters Into which Sewage is Discharged (1913 Partial).
724	9.40	29	Neutralization of Acid Wastes (1913 Partial).
745	12.00	27	Methods of Treating Slaughter House Wastes (1914 Partial).
761	3.40	9	Sewage Screening at Cologne, Germany (1914 Partial).

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SANITATION—(Continued)

No.	Cost.*	No. of Ref.	
784	$9.80	93	Electrical Processes for the Purification of Sewage and Water (1914 Partial).
801	9.40	34	The Effect of Salt Water and Sunlight in Destroying Sewage (1914 Partial).
814	2.80	4	Effect of Scour on Sewers and Sewer Tunnels (1914 Partial).
836	1.00	4	Design of Slaughter Houses and Disposal of Wastes (1914 Partial).
844	2.60	13	Tests of Reinforced Concrete Sewer Pipe (1915 Partial).
850	7.80	67	Municipal Collection and Disposal of Garbage and Ashes (1915 Partial).
867	4.50	54	Methods and Results of Testing Concrete Vitrified Clay Pipe for Resistance to Internal Pressure and External Loading (1915 Partial).
868	4.50	21	Vitrified Clay Pipe (1915 Partial).
869	2.80	5	Clay Pipe (1915 Partial).
917	5.10	33	The Building and Maintenance of Modern Construction Camps (1915 Partial).
922	2.90	5	Sewage Disposal for a Small Seaside Town with a Low Elevation (1916 Partial).
924	4.50	17	Design of Inverted Siphons (1916 Partial).
950	5.20	11	Sewage Pumping Plants of a Capacity of 30 000 000 Gal. and More Per Day (1916 Partial).
966	2.90	3	Cities in the United States Collecting Ashes and Selling them Delivered (1916 Partial).
979	3.90	10	Design of Superstructures of Sewage Disposal Plants for Residential Districts (1916 Partial).

STRUCTURAL

1	0.50	5	Modern Steel-Frame High Buildings (1903 Partial).
9	1.00	17	Sea Water for Mixing Concrete (1901 Partial).
10	1.50	11	German and French Tests and Specifications for Portland Cement (1902 Partial).
11	3.30	9	Comparative Tests of Portland Cement Made at Different Temperatures (1902 Partial).
12	0.50	5	Tests of Cement Mortar Mixed with Sand in Different Proportions (1902 Partial).
13	0.00	7	Laying Concrete Mortar in Cold Weather: Search in *Transactions*, Am Soc. C. E. (1902 Complete).
38	3.50	15	Pile-Driving (Iron and Steel) (1901 Partial).
67	0.75	5	Effects of Sea Water on Cement (1903 Partial).
70 70a 70b	8.60	74	Concrete Piles (1908 Partial).
76	0.50	7	Disposition of Structures to Meet Aesthetic Considerations (1904 Partial).
79	0.50	11	Grain Elevators (1904 Partial).
81	1.25	6	Keene's Cement (1904 Partial).
90	5.50	96	Coffer-Dams and Deep Foundations (1904 Partial).
91	2.00	23	Protected Steel Columns in Fires (1904 Partial).
93	2.00	12	Collapse of Buildings (1904 Partial).
94	1.00	7	Comparative Merits of Cast-Iron and Steel Columns (1904 Partial).
95	1.00	16	Cast-Iron Column Tests (1904 Partial).
96	5.00	86	Concrete-Metal Construction (1904 Partial).
98	1.50	24	Forms for Concrete Masonry (1904 Partial).
100	2.50	20	Water Jet for Sinking Caissons and Piles (1904 Partial).
105	1.75	6	Specifications for Paint for Railway Bridges and Buildings (1904 Partial).
108	2.50	18	Comparative Test of Various Paints (1904 Partial).
112	2.00	12	Durability of Wood Under Water: Data and Statistics (1904 Partial).
116	2.50	36	Steel Construction for Large Buildings (1904 Partial).
133	1.40	7	Wakefield Sheet-Piling (1905 Partial).
147	1.25	1	Iron Pipe in Structural Work (1905 Partial).
151 151a	7.65	51	Injection of Cement in Water-Bearing Strata (1913 Partial).
154 154a	10.50	89	Disintegration of Concrete by Sea Water (1914 Partial).

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STRUCTURAL—(Continued)

No.	Cost.*	No. of Ref.	
175	$2.00	8	Steel-Concrete Construction (Papers by A. L. Johnson) (1906 Partial).
176	2.00	13	Steel or Expanded Metal Used in Walls of Concrete to Prevent Cracking (1906 Partial).
179	4.25	47	Concrete-Steel Construction (1906 Partial).
189	4.50	26	Concrete Floors for Loads of More Than 250 Lb. per Sq. Ft. (1906 Partial).
192	1.00	16	Design and Requirements of Horse Stables (1906 Partial).
195	10.75	87	Corrosion of Iron and Steel (1906 Partial).
207	3.95	6	Methods Used by the Romans in Making Mortar or Concrete (1906 Partial).
208	3.95	7	Analysis of Puzzolanic Cements (1906 Partial).
221	13.05	72	Placing of Concrete under Water (1906 Partial).
227	7.75	15	The Use of Broken or Crushed Brick as an Aggregate in Concrete (1906 Partial).
231 231a 231b	10.40	98	Reinforced Concrete Columns (1912 Partial).
234	3.38	17	Tests of Various Forms of Steel to Find the Modulus of Elasticity (1906 Partial).
235	1.50	1	Tests of Twisted Bars of Steel for Elongation (1906 Partial).
236	3.38	20	Comparative Tests of Different Systems of Reinforced Concrete (1906 Partial).
242	5.40	18	Deterioration of Steel, Wood, or Masonry Structures (1906 Partial).
247	3.00	27	Crazing of Cement (1906 Partial).
250	4.95	15	Illustrations Showing Layout and Floor Plans of Hospital Buildings (1907 Partial).
252	1.00	3	Cypress Wood for Piles (1907 Partial).
257	1.15	7	Specifications, Methods, etc., for Building Water-proof Reinforced Concrete Reservoirs (1907 Partial).
259	5.75	55	Failure of Reinforced Concrete Structures (1907 Partial).
278	5.85	15	Impact Tests (1907 Partial).
280	1.15	20	Fire-proofing of Floor Construction (1907 Partial).
281	4.85	26	Adaptability or Suitability of Long-Span Concrete Floor Construction, Use of Cinder-Concrete for the Same, together with *Ec* and Working Stresses Allowable (1907 Partial).
282	1.80	1	Increased Efficiency of Haunched I-Beams in Reinforced Concrete Floor Construction (1907 Partial).
303	1.60	14	Methods, Uses, and Cost of Creosoting Timber (1907 Partial).
308	4.00	38	Riveted Beam Connections and Riveted Splices (1907 Partial).
321 321a	10.40	93	References in the *Official Gazette* of the United States Patent Office to Reinforced Concrete Poles and Columns (1914 Partial).
328	2.25	4	Manufacture of Seagliola or Imitation Marble (1907 Partial).
338 338a	3.20	25	Settlement Caused by Underground Works (1909 Partial).
339	12.40	93	Use of Asphalt and Coal-Tar for Pipe-Dips, Metal Coatings, and Water-Proofing (1908 Partial).
345	5.75	52	Fire-proof Coal Pockets and Grain Elevators (1908 Partial).
346	4.15	38	Decorative or Artistic Use of Structural Steel (1908 Partial).
355 355a	10.90	96	Metal Sheet-Piling (1910 Partial).
389	4.40	18	Coverings for Reinforced Concrete Factory Floors (1908 Partial).
412	2.50	35	List of American Periodicals on Cement, Concrete, and Reinforced Concrete (1909 Partial).
413	2.35	13	Relative Value of Sand *vs.* Broken Stone Screenings in Cement and Concrete Work (1909 Partial).
414	12.25	51	Experiments on Wrought-Iron or Steel Compression Members, Compiled for the Special Committee on Steel Columns and Struts (1909 Partial).
420	4.50	44	Sinking Open Wells or Caissons by Processes Other than Pneumatic (1909 Partial).
442 442a	6.55	52	Test Borings for Foundations (1911 Partial).
443	20.15	86	Quicksand (1909 Partial).
446	41.65	142	Reinforced Concrete Industrial Buildings (1909 Partial).
448	1.80	20	Books on Structural Mechanics and Theory of Framed Structures by German Authors (1909 Partial).

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STRUCTURAL—(Continued)

No.	Cost.*	No. of Ref.	
449	$4.50	36	Driving Metal Sheet-Piling (1909 Partial).
450	4.75	10	Effect of Coal Gases and Constant Heat on the Life of Steel and Concrete when Unprotected (1909 Partial).
457 457a	19.95	164	Water-proofing of Masonry Structures (1910 Partial).
458	4.50	17	Sand Cement (1909 Partial).
471	3.50	8	Foundations in Chicago (1910 Partial).
478	1.25	2	Foundations of the St. Paul Building, New York City (1910 Partial).
483a 483b	26.15	125	Marine Wood Borers (Includes Search No. 483) (1912 Partial).
488	2.65	10	Use of Tower Hoisting Arrangement and Spout or Conduit for Depositing Concrete, Prior to 1907 (1910 Partial).
495	5.65	70	Method used to Reinforce Large Structures, Subject to Settlement (1910 Partial).
500	2.60	2	Oil Paint for Roundhouses (1910 Partial).
503	0.75	2	Deposition of Concrete through Pipes, by Compressed Air (1910 Partial).
509	6.55	18	Action of Sulphur Water on Cement and Concrete (1910 Partial).
511 511a 511b	8.95	35	Mushroom System of Reinforced Concrete Construction (1912 Partial).
516	7.50	60	Wood Piles (1910 Partial).
517	4.00	64	Preservation of Timber (1910 Partial).
534	5.20	12	Piles About 80 Ft. in Length (1911 Partial).
535	3.60	19	Tests of Sand-Lime Brick (1911 Partial).
537	4.90	22	Failures in Reinforced Concrete and Steel Buildings (1911 Partial).
539	6.50	34	Concrete Piles and Allowable Loads on Them (1911 Partial).
540	4.25	5	Tanks of Concrete, Clusters or Single (1911 Partial).
542	4.25	16	Secondary Stresses in Framed Structures (1911 Partial).
552 552a	12.90	28	Foundations of High Buildings Constructed by the Water-Tight Dam Process (1911 Partial).
556	5.20	21	Disintegration of Concrete Structures in Sea Water Exposed to Alternate Thawing and Freezing (1911 Partial).
565	3.90	30	Gas Holders: Construction and Cost (1911 Partial).
568	8.80	42	Wooden Piles Coated with Concrete (1911 Partial).
569	4.07	11	Use of Crushed Blast Furnace Slag as an Aggregate in Portland Cement Concrete (1911 Partial).
570	4.10	22	Steel Wire and Wire Products (1911 Partial).
573 573a	15.65	29	Concrete Products (1911 Partial).
577	42.10	83	Methods of Determining Voids in Broken Stone or Sand (1911 Partial).
579	3.60	14	Sinking or Driving Steel Foundation Cylinders by any Other Method than the Pneumatic Process (1911 Partial).
584	1.65	9	Painting Concrete Structures (1911 Partial).
600	17.55	31	Corrosion of Iron and Steel in Cinder Concrete (1912 Complete).
603	7.00	66	Methods of Concreting in Cold Weather (1912 Partial).
604	10.10	39	Open Well Foundations for Buildings (1912 Partial).
606	11.55	27	Earth Pressures (1912 Partial).
615	17.90	2	Resistance of Concrete or Stone Columns, Laterally Reinforced (1912 Partial).
618	4.05	4	Data on Fabricated Structural Steel Bridges and Buildings in the United States (1912 Partial).
619	11.55	27	Driving Reinforced Concrete Piles, and Comparison between Steam and Drop-Hammer Drivers (1912 Partial).
621	9.75	5	Neat Cement Finish for Floors (1912 Partial).
622	7.15	1	Concrete Tanning Vats (1912 Partial).
624	8.60	13	Thermo-Electric Determination of Stresses (1912 Partial).
629	3.90	15	The Cement Gun (1912 Partial) (*See also* Search No. 693).
635 635a	26.00	7	Structures in which the Floor Slab is Integral with the Belt Course Around the Wall (1912 Partial).
644	8.65	14	Durability of Steel in Earth, Varying from Damp to Wet (1912 Partial).
648	0.70	5	Depositing Concrete Under Water by Means of a Tremie (1911 Partial).
655	6.85	12	Comparison between Steam and Drop-Hammer Pile-Drivers (1912 Partial).

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STRUCTURAL—(Continued)

No.	Cost.*	No. of Ref.	
658	$4.25	21	Moving of Masonry Buildings without Demolition (1912 Partial).
661	3.60	3	Flag Poles for Tall Buildings (1912 Partial).
663	1.45	12	Reinforced Concrete Grand Stands (1912 Partial).
670	5.20	32	Physical Properties of Nickel Steel (1913 Partial).
671 671a	38.85	65	Prevention of Corrosion of Steel Encased in Concrete (1913 Partial).
672	4.25	5	Settlement of the Foundations of the Leaning Tower of Pisa, Italy (1913 Partial).
679	3.60	28	Action of Salt Water on Concrete (1913 Partial).
684	13.45	36	Cement Paint (1913 Partial).
688	7.15	31	Grain Elevators (1913 Partial).
691	10.90	48	Effect of Low Temperatures on the Strength and Ductility of Iron and Steel (1913 Partial).
693	4.70	26	Means of Depositing Concrete and Grout by Compressed Air, Including the Cement Gun (1913 Partial).
695	4.55	24	Public Markets (1913 Partial).
698	2.95	6	Damages to Buildings by Excavation of Rock in their Vicinity, and Vibration of Buildings (1913 Partial).
708	4.20	7	Paints and Washes for Concrete Surfaces (1913 Partial).
713	6.50	41	Corrosion of Iron and Steel in Concrete (1912 Partial).
714	3.60	11	Accelerated and Steam Pressure Tests of Cement, 1893-1913 (1913 Partial).
722	4.70	20	Distribution of Concrete by Gravity (1913 Partial).
723	3.40	10	Kyanizing Process of Treating Timber (1913 Partial).
725	2.00	2	German Method of Designing the Columns in a Workshop Building (1913 Partial).
733	10.40	71	Effect of Foul and Acid Waters and Alkali on Concrete and Cement (1913 Partial).
737	12.20	47	Behavior of Buildings of Reinforced Concrete in Severe Fires (1913 Partial).
741	9.90	43	Failures of Structures either not Designed or not Inspected by an Engineer (1913 Partial).
744	7.30	16	Retempered or Regauged Cement Mortar or Concrete (1914 Partial).
746	6.00	23	Placing Concrete by the Pneumatic Process (1914 Partial).
758	3.10	10	Timber Embedded in Concrete (1914 Partial).
764	100.25	856	Reinforced Concrete (1914 Partial).
769	2.00	2	Production of Steel Sheet-Piling in the United States (1914 Partial).
770	2.00	3	Cotton Warehouses and Appliances (1914 Partial).
773	2.90	21	The Use of Broken Stone Screenings instead of Sand for Cement Mortar and Concrete (1914 Partial).
775	5.20	53	Design of Gas Holders (1914 Partial).
778	3.70	11	Stresses Produced in Structural Steel Building Frames due to Oscillating Motion (1914 Partial).
781	5.00	19	Theory of Combined Stresses in Structural Steel Members, Subject to Tension and to Transverse Loads at the Same Time (1914 Partial).
783	10.10	100	Rust-Proof Paints (1914 Partial).
789	6.20	18	Settlement of Streets and Buildings due to the Construction of Tunnels, in America (1914 Partial).
793	7.60	51	Borings for Investigating Foundations of Bridge Piers and Buildings (1914 Partial).
796	5.90	41	Bearing Power of Soils (1914 Partial).
798	9.75	68	Impermeability of Concrete: Integral Method of Water-proofing Concrete and Cement Mortar (1914 Partial).
808	6.30	27	Concrete Tenement Houses and Factory Workers' Cottages (1914 Partial).
818	1.00	8	Wind Bracing in Buildings (1914 Partial).
822	6.20	7	Stockyard Design (1914 Partial).
825	25.00	184	Manufacture and Use of Concrete Piles (1914 Partial).
827	4.70	12	Test Loads at which First Crack Appears in Reinforced Concrete Slabs and Beams (1914 Partial).
829	5.20	19	Effect of Adding Hydrated Lime to Cement Concrete (1915 Partial).
833 833a	9.20	46	Extra Work in Contracts (1915 Partial).
834	3.90	6	Condition of Wood Under Ground or Under Water for More than 25 Years (1915 Partial).

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STRUCTURAL—(Continued)

No.	Cost.*	No. of Ref.	
853	$2.00	13	Manufacture of Slag Brick (1915 Partial).
855	9.80	83	Action of Salt Water on Reinforced Concrete, Concrete, and Cement (1915 Partial).
859	0.00	3	Distribution of Concentrated Load through Deep Earth Filling (1915 Partial).
861	9.90	28	The Fabrication and Use of Concrete Sheet-Piles in Salt Water (1915 Partial).
862	1.00	9	Proportioning and Strength of Cinder Concrete (1915 Partial).
866	12.70	76	Plants for the Mining, Pumping, Washing, Screening, and Drying of Sand (1915 Partial).
872	1.50	3	Berquist Bunkers and Brownhoist Suspended Concrete Bins (1915 Partial).
873	3.60	6	Failures of Brick Chimneys more than 50 Ft. High (1915 Partial).
874	3.40	4	Number of Concrete Chimneys Erected in the United States and Canada (1915 Partial).
875	4.70	8	Temperatures in Chimneys used in Connection with Incinerators (1915 Partial).
876	5.90	19	Arguments Opposed to the Statement that Concrete Chimneys are More Likely to Fail than Brick Chimneys (1915 Partial).
897	2.60	4	Specifications for Rip-rap (1915 Partial).
899	5.20	18	Cracks in Plain and Reinforced Concrete Caused by Changes in Temperature (1915 Partial).
903	9.90	8	Construction of Masonry on Embankments (1915 Partial).
907	3.30	37	Life of Iron and Steel Structures (1915 Partial).
910	11.70	15	Use of Timber Sawdust, Asbestos, or Cork in Concrete (1915 Partial).
911	19.80	7	Safe Loads for Wooden Piles More Than 50 Ft. Long, in Mud Bottom (1915 Partial).
912	2.90	14	Cotton Storage and Handling (Includes Search No. 770) (1915 Partial).
919 919a	6.80	29	Methods of Screening and Drying Sand (1916 Partial).
920	9.10	44	Definitions of Reinforced Concrete (1916 Partial).
928	5.00	11	Water-proofing and Hardening of Concrete Floors (1916 Partial).
929	4.40	29	Methods of Shoring, Bracing, and Sheathing Trenches (1916 Partial).
932	5.90	19	Construction of Fire-Places and Flues (1916 Partial).
933	6.50	26	Invention of Reinforced Concrete (1916 Partial).
934	5.50	29	Workmen's Cottages in Connection with Industrial Plants (1916 Partial).
944	9.90	14	Pre-Cast Concrete Sections Used Prior to 1900 (1916 Partial).
954 954a	11.20	53	Tests of Brick and Hollow Tile in Walls and Piers' (1916 Partial).
956	4.70	18	Dry Masonry Construction (1916 Partial).
958	1.10	1	Formula for Finding Percentage of Spiral Steel in Hooped Columns of Reinforced Concrete (1916 Partial).
959	5.60	12	Tremie Method of Depositing Concrete in Deep Water (1916 Partial).
984	1.00	4	Earth Pressures (1916 Partial).
985	2.10	3	Measurement of Concrete Work in the United States and Canada (1916 Partial).
989	1.10	2	Agreement of the Düsseldorf German Wire Union (1916 Partial).

TOPOGRAPHICAL

36	1.00	3	Magnetic Declination in Mexico (1903 Partial).
68	0.50	4	Photographic Surveying (1903 Partial).
158	1.00	5	Methods of Treating Street Intersections (1905 Partial).
809	2.45	7	Land Surveys in the Southern Appalachian Mountains (1914 Partial).
810	1.30	3	Official Survey of Roanoke, Va., Made in 1891 and 1892 (1914 Partial).
826	2.70	2	Smith Solar Transit (1914 Partial).
937	4.90	15	Topographic Signs (1916 Partial).

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WATER SUPPLY AND WATER POWER

No.	Cost.*	No. of Ref.	
17	$2.50	15	Crib Dams (1901 Partial).
64 } 64a }	5.65	21	Flow of Water in Brick Conduits (1910 Partial).
77	1.25	13	Steel-Concrete and Wood-Stave Pipe Conduits (1904 Partial).
82	1.50	20	Intake Tunnels at Cleveland and Chicago (1904 Partial).
85	2.50	32	Recent Developments of Water Powers of Large Size and High Heads; April, 1904 (1904 Partial).
86 } 86a }	3.75	16	Flow of Water in Concrete Channels (1910 Partial).
99	1.05	4	Deterioration of Cast-Iron Pipe (1904 Partial).
103	5.25	17	Effect of Storage of Ground-Water in Open Reservoirs (1904 Partial).
107	5.40	11	Cost of Operation, etc., of Power Plants (1904 Partial).
114	1.80	3	Description of Great Northern Paper Co. Plant at Millinocket, Me. (Water Power) (1904 Partial).
117	4.25	36	Appraisal and Valuation of Water-Works (1904 Partial).
123	5.00	105	Concrete Construction before 1897 (Dams and Hydraulic Work) (1904 Partial).
132	1.50	5	Right of Eminent Domain (Water-Works) (1905 Partial).
134	1.40	8	Consumption of Water per Capita (1905 Partial).
143	5.50	84	Electrolysis of Water Mains (1905 Partial).
149	3.00	13	Methods of Laying Concrete in Dams 40 ft. High and Greater (1905 Partial).
153 } 153a }	12.25	66	Cost of Development and Rental of Water Power in North America (1909 Partial).
166	0.75	10	Water Towers (1905 Partial).
167	6.00	51	Small Water-Power Plants of the Past 8 Years, 1897–1905 (1905 Partial).
168	2.50	21	Sand Washing and Pumping (1905 Partial).
172	2.25	15	Construction and Operation of Water-Conduit Tunnels (1905 Partial).
183	5.00	31	Rock-fill dams (1906 Partial).
184	59.00	282	Water Waste Prevention (1906 Partial).
194	10.50	64	Duty of Water and Loss of Water from Canals and Reservoirs (1906 Partial).
196	1.75	12	Rainfall in Mexico (1906 Partial).
200	1.25	13	Utilization of the Ebb and Flow of Tide for the Generation of Power (1906 Partial).
203	23.85	106	"Bottom of a Dam" and "Head" of Water on a Dam; References Tending to Make Clear the Meaning of the Terms (1906 Partial).
204	1.50	18	Utilization of the Rise and Fall of Tide for the Generation of Power (1906 Partial).
205	7.20	35	Life of Cast-Iron, Cement, Wrought or Riveted-Steel Pipes (1906 Partial).
206	1.50	14	Utilization of the Power of Ocean Waves (1906 Partial).
210	1.35	8	Submerged Intakes, Principally of Iron and Steel (1906 Partial).
214	5.65	18	Reinforced Concrete Conduits and Penstocks 8 Ft. in Diameter and Greater (1906 Partial).
216	4.65	70	Water-Power Developments (1906 Partial).
219	4.05	21	Water-Power Development, 4 000 H. P. and Greater (1904–1906) (1906 Partial).
222	3.00	22	Design, Construction and Cost of Large Flumes (1906 Partial).
228	4.05	51	Failures of Dams During the 5 Years Ending October, 1906 (1906 Partial).
239	3.60	15	Tests of Cement Pipe (1906 Partial).
246	7.65	112	Water Purification; Methods and Maintenance (1906 Partial).
254	3.60	21	Description of Hydro-Electric Power Plants of Less than 100 Ft. Head on Mississippi River and Tributaries (1907 Partial).
263	1.80	3	Cost of Engineering in Hydro-Electric Developments (1907 Partial).
264	3.25	24	Cost and Methods of Construction of Reinforced Concrete Conduits (1907 Partial).
269	4.85	30	Theory and Process of Operating Siphons (1907 Partial).
270	2.25	8	Construction of Flash-Boards on Dams (1907 Partial).
271	1.50	17	Undeveloped Water Powers in the United States (1907 Partial).
274	1.00	10	Experiments on the Flow of Water over an Ogee Dam (1907 Complete).

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WATER SUPPLY AND WATER POWER—(Continued)

No.	Cost.*	No. of Ref.	
275	$1.65	16	Back-Water Due to the Construction of Fixed Dams (1907 Complete).
284	5.40	28	On the Use of Underground Water from the Beds of Rivers (1907 Partial).
288	0.75	11	Water-Works Systems for Cities, 1899–1907 (1907 Partial).
291c	8.80	56	Valuation of Water Power, Particularly with Reference to Condemnation Proceedings (Includes Nos. 291, 291a, and 291b) (1909 Partial).
293	2.00	15	Work and Plans of the New York Board of Water Supply (1907 Partial).
294	2.50	2	Details of Construction of Works of the Pike's Peak Hydro-Electric Co.; Especially the Handling, Laying and Securing of the Pipes (1907 Partial).
315	0.50	7	San Carlos Project for Storage of Water on Gila River, Arizona (1907 Partial).
348	4.05	12	Tongued and Grooved Sheet-Piling as a Cut-off Wall for Dams (1908 Partial).
360	5.40	47	Water Softening (1908 Partial).
362	16.80	106	Gate-Houses, Valves, and Screens for Masonry and Earthen Dams (1908 Partial).
369	2.95	5	Methods of Keeping Canals Free from Deleterious Growths of Weeds (1908 Complete).
376	21.05	34	Large Dams or Retaining Walls that Impound Large Bodies of Water above Cities or Villages (1908 Partial).
380	2.05	5	Relation of Rainfall to Run-Off on the Water-Shed of Rock River, Ill. (1908 Partial).
381	6.30	53	Reinforced Concrete Conduits 5 Ft. or Less in Diameter (1908 Partial).
384	2.70	5	Inverted Siphons for Water Supply, Constructed in Rock (1908 Partial).
401	9.60	35	Large Siphons (1908 Partial).
408	1.80	6	Junction of Earth-Sections with Lined Sections or Flumes in Canal Construction (1909 Partial).
410	3.25	21	Conditions which Cause Stored Waters to have Fish and Oil or other Bad Tastes (1909 Partial).
411	3.15	8	Contamination of Water for City Use by Alternate Raising and Lowering of the Surface Level of Water in Ponds (1909 Partial).
415 415a 415b	29.30	82	Awards for Water or Water-Power Diversion (1909 Partial).
416	4.05	22	Deep-Well Pumping by Compressed Air (1909 Partial).
433	0.60	3	Irrigation in Texas (1909 Partial).
440	0.90	9	New Orleans Sewerage and Water-Works Systems (1909 Partial).
456	11.75	78	Cast-Iron Water Pipe *versus* Steel, Wrought-Iron and Wooden Pipe (1909 Partial).
459	5.15	5	Animal Life in Water Supply (Snails) (1909 Partial).
465	3.50	17	Rice Irrigation in the United States (1909 Partial).
473	2.25	10	Wooden Stave Pipe, Crossing Rivers (1910 Partial).
485	2.75	13	Conduits through Earthen Embankments (1910 Partial).
494	4.00	14	Methods of Laying Submerged Cast-Iron Mains, Subject to Pressure (1910 Partial).
499	5.50	20	Spillways for Earth Dams (1910 Partial).
501	1.00	1	Preservative Coatings for Steel Mains (1910 Partial).
502	4.25	13	Specifications for Laying Cast-Iron Water Mains, about 30 In. in Diameter (1910 Complete).
506	3.75	11	Storage of Water for Logging Purposes (1910 Complete).
521	7.40	7	Settlement of Timber Dams on Rock Foundations (1910 Partial).
522	14.30	57	Reinforced Concrete Stand-pipes (1910 Complete).
524	8.94	25	Flow of Water in Cement-Lined Circular Conduits and Riveted Steel Pipe, 6 Ft. or More in Diameter (1910 Partial).
538	1.65	9	Methods and Machinery for Leveling and Grading Land for Irrigation (1911 Partial).
543	4.05	23	Rates for Water Furnished by Private Companies for Domestic Use (1911 Partial).
544	3.90	36	Underground Water from Tubular Wells as a Source of Public Water Supply (1911 Partial).
548	2.60	5	Depositing the Silt from Fresh-Water Streams on Land (1911 Partial).
550	4.90	53	Power Inherent in Tidal Waters (1911 Partial).

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WATER SUPPLY AND WATER POWER—(Continued)

No.	Cost.*	No. of Ref.	
555	$2.30	4	Municipal Hydrant Rental for General Fire Protection Purposes (1911 Partial).
561	2.60	11	Transportation of Silt in Irrigation Canals and Channels (1911 Partial).
562	3.25	16	Water-Logging of Irrigated Lands by Absorption and Percolation (1911 Partial).
567	4.55	3	Data Relating to the Croton and Wachusetts Water-Sheds (1911 Partial).
571	6.85	23	Irrigation Canals, Capacity 1 000 Cu. Ft. per Sec. and Greater (1911 Partial).
575	5.35	3	Filtration of Salt Water (1911 Complete).
582	7.00	26	Meters for Irrigation Systems (1911 Partial).
585	3.25	15	Water Finders (1911 Partial).
590	0.50	3	The Purification of Water by Electricity (1911 Partial).
592	6.20	35	Steel Dams (1911 Partial).
602	2.15	6	Irrigation in Porto Rico (1912 Complete).
607	1.80	1	Specifications for Turbine, Water-Wheel Installations for Power Plants (1912 Partial).
630	13.65	20	Amount of Leakage under Pressure from Cast-Iron, Concrete, and Wood Stave Pipe (1912 Partial).
632	1.95	6	Resistance of Earthen Dams and Embankments against Water Pressure (1912 Partial).
636	12.70	3	Earliest Use of the Block System of Rates for Water Supply (1912 Complete).
638	14.30	9	The Merits of Inserting Valves of Smaller Diameter than the Main itself in a Water Main (1912 Partial).
640	10.25	8	The Utilization of Water-Power, Heads of 3 Ft. and Lower, as a Source of Electrical Energy (1912 Partial).
650	2.60	3	Siphon at Mittersheim (1912 Partial).
652	7.30	50	California Development Company, Rio Grande and Salt River Irrigation Projects (1912 Complete).
653	3.90	14	Duty of Water in the Irrigation of Sugar Cane (1912 Partial).
674	14.00	20	Deposits of Gravel and other Material in Storage Reservoirs in Rivers (1913 Partial).
694	3.05	85	Water Meters (1913 Partial).
697	2.30	3	Flow of Water over a Dam the Crest of which is Oblique to the Flow of the Stream (1913 Partial).
699	10.10	8	Coefficient of Roughness in Steel Pipe 6 Ft. and Greater in Diameter (1913 Partial).
700	1.45	3	Leadite for Water Pipe Joints (1913 Partial).
707	3.40	9	Drainage and Irrigation of Salt Marsh (1913 Partial).
740	9.30	7	Valuing Land for Water Power (1913 Partial).
743	9.90	41	Laws or Practice Under Which Water Storage may be Developed in Europe (1913 Partial).
747	17.05	81	Construction of Steel Pipe (1914 Partial).
748	17.05	50	Durability of Steel Pipe (1913 Partial).
771	5.35	12	Increasing the Height of Earthen Dams (1914 Partial).
784	9.80	93	Electrical Processes for the Purification of Sewage and Water (1914 Partial).
791	13.00	120	Dam Failures (1914 Partial).
795	1.00	4	Chucharas Dam No. 5 of the Pueblo-Rocky Ford Irrigation Co., Colorado (1914 Partial).
804	7.30	25	Leakage in Cast-Iron Water Pipe (1914 Partial).
811	1.95	3	Water-Hammer. Articles by Lorenzo Allievi (1914 Partial).
819	3.00	10	Construction of Dams on Porous Rock Foundations, and Methods Used for Making Them Water-Tight (1914 Partial).
824 824a	19.70	195	Manufacture and Use of Concrete Pipe (1914 Partial).
830	7.60	35	Water Supply of European Cities with a Population of 25 000 or More Having Well Water Supplies (1915 Partial).
831	3.90	13	Cost-Keeping Systems as Applied to Irrigation Projects (1915 Partial).
838 838a	5.40	18	Design of Reinforced Concrete Aqueducts More than 5 Ft. in Diameter (1915 Partial).
846	1.80	12	Reduction in Domestic Water Consumption Following the Universal Installation of Water Meters (1915 Partial).
847	6.00	49	Municipal Hydrant Rental from Private Water Companies (1915 Partial).
877	4.70	5	Derivation of Formula for Finding Time of Lowering Water in a Reservoir without Regard to Inflow or Evaporation (1915 Partial).

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WATER SUPPLY AND WATER POWER—(Continued)

No.	Cost.*	No. of Ref.	
887	$8.90	10	The Influence of Forests on Annual Rainfall (1915 Partial).
891	22.50	50	Seepage in Rock Foundations of Dams and Reservoirs (1915 Partial).
892	4.60	18	The Infiltration of Water Through Sand and Gravel (1915 Partial).
894	3.70	11	Use of Oils or Asphalts in Puddle Mixtures for Stopping Seepage in Reservoirs and Canals (1915 Partial).
900	4.90	12	High-Pressure Tunnels Used for Hydro-Electric Development, and for Water Supply (1915 Partial).
901 901a	10.30	15	Behavior of Cast-Iron Pipe Under Stresses Due to External Loading (1915 Partial).
905	2.90	11	Irrigation by Pumping from Deep Wells (1915 Partial).
924	4.50	17	Design of Inverted Siphons (1916 Partial).
940	9.90	8	Coefficient of Roughness for Tunnels Constructed Through Rock (1916 Partial).
957	6.70	9	Laws Regulating Underground Water (1916 Partial).
963	8.45	18	Bridge Foundations in the Mississippi River (1916 Partial).
964	10.40	7	Corrosion and Pitting in the Runners of Water Wheels (1916 Partial).
965	10.10	34	Growth in, and Tuberculation of, Cast-Iron Water Mains (1916 Partial).
968	2.30	24	Fish Screens or Fishways for Dams (1916 Partial).
974	7.10	10	Reinforced Concrete Lock-Joint Pipe (1916 Partial).
983	3.70	9	Definition of the Word Flashboard (1916 Partial).

WATERWAYS

5 5a	6.70	54	Canal Lock Gates (1914 Partial).
6	1.00	9	Traffic, Tolls Charged, etc., in the Suez, North Sea, and Baltic Canals (1901 Partial).
21	4.00	46	Hydraulic Dredges or Sand Pumps (1903 Partial).
23	2.00	25	Dikes, Low and Movable Dams (1901 Partial).
31	1.00	3	Jetty Harbors of the Baltic Sea (1903 Partial).
34	0.70	7	South Pass Jetties of the Mississippi (1901 Partial).
37	1.00	9	Current Meters (1903 Partial).
61	3.50	53	Shore Protection, Wave Action, Ocean Currents, Jetties, etc. (1903 Partial).
69	0.70	5	Canals and Canal Locks (1903 Partial).
72	3.75	39	Lock Gates for Tidal Harbors (1903 Partial).
80	1.35	19	Canal Construction (1904 Partial).
102	1.25	17	Coal and Ore Docks in the West (1904 Partial).
131	2.75	2	Breakwater at Cherbourg (1905 Partial).
136	1.85	3	Amount and Cost of Excavation at the "Iron Gates" of the Danube River (1905 Partial).
137	3.15	21	Atchafalaya River: Objects, Methods, Cost and Success of the Measures Taken to Sink Sills at the Entrance (1905 Partial).
141	0.75	4	References in Reports of Chief of Engineers, U. S. Army: Locks, Dams and Improvements on the Ohio, Monongahela and Allegheny Rivers under Major W. L. Sibert; and on the Ohio, Kanawha, and Little Kanawha Rivers in West Virginia under Major G. A. Zinn; Survey of Illinois and Desplaines Rivers, Ill., for Waterway from Lockport, Ill., to St. Louis, Mo. (1905 Partial).
161	5.00	48	Evaporation in Natural Reservoirs (1905 Partial).
193	1.00	20	The Elbe-Trave, Teltow, and Oder-Spree Canals (1906 Partial).
199	6.50	66	Hydraulic Dredges, Depositing Material on Shore through Floating Pipe Lines (1906 Partial).
202	1.50	16	Jetties and Breakwaters in Harbors (1906 Partial).
209	2.00	13	Water Supply for Canals (1906 Partial).
213	5.40	27	Timber Wharves Constructed in Sea Water (1906 Partial).
255	2.00	11	Cape Cod Canal (1907 Partial).
268	4.25	6	Data Regarding the Coefficient of Friction to be Used in Calculating Resistance of Stone-Filled Timber Cribs against Sliding on Earth, Rock, or Gravel Foundations (1907 Partial).
273	3.15	9	Effect of Ice Fields on Dams and Abutments (1907 Partial).
276	2.00	21	Fishways (Includes No. 190) (1907 Partial).

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WATERWAYS—(Continued)

No.	Cost.*	No. of Ref.	
286	$1.25	13	Harbors Constructed on Sites with Open Roadsteads and Low Sandy Beaches (1907 Partial).
295	7.15	74	Mechanical Towage of Canal Boats Outside of the United States (1907 Partial).
296	2.50	7	Dikes for Narrowing and Rectifying a Broad Sandy Stream (1907 Partial).
299	1.45	3	Chicago Drainage Canal (1907 Partial).
300	0.80	6	Manchester Ship Canal (1907 Partial).
301	1.15	6	Wharves, Bulkheads, etc. (1907 Partial).
312	4.40	20	Ocean Piers in the United States and Foreign Countries (1907 Partial).
335	0.50	1	Columbia River, Oregon, Canal around the Cascades (1907 Partial).
353	4.30	57	Improvement of the Mississippi and Missouri Rivers (1908 Partial).
354	3.50	28	Floods and the Relation of Forests to Stream Flow in European Countries (1908 Partial).
363	19.55	31	Wooden Piles with an External Covering of Concrete Encased in Metal, Wood, or Earthenware (1908 Partial).
377	6.30	8	Seepage in Canals (1908 Partial).
382	3.85	42	Dredging in Harbors in Sand and Mud, Apparatus and Cost of Operation (1908 Partial).
388	6.30	84	Levees (1908 Partial).
390	4.50	23	Shore Protection by Lining with Concrete (1908 Partial).
391	3.50	2	Foundations on Coral Reefs (1908 Partial).
394	2.35	30	Proposed Lighthouse on Diamond Shoals Off Cape Hatteras (1908 Partial).
395	3.95	47	Government Publications on Lighthouses (1908 Complete).
396	1.15	17	Lighthouses (1908 Partial).
402 402a	9.65	84	Dock Construction (1911 Partial).
405	0.50	6	Galveston Sea-Wall (1909 Partial).
422	3.72	11	The Port of Philadelphia (1909 Complete).
423	3.73	17	The Port of Hamburg (1909 Complete).
430	0.70	4	The Southwest Pass of the Mississippi River (1909 Partial).
432	1.35	7	Sabine Pass (1909 Partial).
434	0.50	4	Improvement of Charleston Harbor (1909 Partial).
439	1.35	4	The Chalmette Docks of the New Orleans Terminal Co. (1909 Partial).
451	1.80	3	Barge Canal (1909 Partial).
464	3.50	5	Traffic on the Thames River (1909 Partial).
482	1.90	15	Jamaica Bay, Rockaway Point and Inlet (1910 Complete).
504	9.25	17	Measurement of Dredged Material (1910 Partial).
505	3.50	9	Definition of Rip-rap and Stone Paving (1910 Partial).
515	13.00	81	Articles Containing Illustrations of Cross-Sections of Breakwaters (1910 Partial).
523	10.40	20	Construction of Bulkheads with Foundations in from 40 to 50 Ft. of Silt (1910 Partial).
533	6.50	37	Stream-Flow Data for Streams East of the Mississippi River (1910 Partial).
557 557a	9.85	57	Coal Handling Equipment at Port Terminals (1911 Partial).
563	1.00	8	Bear Trap and Similar Dams (1911 Partial).
564 564a	8.30	26	Scour in Rivers Caused by Erection of Bridge Piers and Methods Used for Their Protection (1911 Partial).
581	14.95	79	Subaqueous Rock Excavation (1911 Partial).
583	9.45	45	Prevention of Scour by Means of Mattresses (1911 Partial).
588	1.65	1	Concrete Sea-Wall at Cebu, Philippine Islands (1911 Partial).
620	10.75	59	The Colorado River (1912 Partial).
634	7.30	27	Harbor Improvements, More Especially Jetties and Breakwaters (1912 Partial).
642	9.55	65	The Substructure of American Docks (1912 Partial).
643	6.50	26	Protection of Timber on the Pacific Coast against Marine Wood-Borers (1912 Partial).
654	7.80	21	The Suez, Kaiser Wilhelm, Manchester, Erie, and St. Marys Falls Canals: Original, Present and Proposed Dimensions (1912 Partial).
662	5.00	26	Raft Chutes at Dams (1912 Partial).
673	3.40	9	Piers with Cast-Iron or Steel Piles (1913 Partial).
706	11.70	25	Steel Lock Gates for Locks 80 Ft. or More Wide (1913 Partial).

* The original cost of the search is given in each case, and copies may be obtained at the price stated. There is a discount of 50% to members of the Society.

WATERWAYS—(Continued)

No.	Cost.*	No. of Ref.	
721	$1.70	3	Drags for Sounding (1913 Partial).
736	11.20	83	Port of New York, Freight Facilities and Statistics (1913 Partial).
752	8.30	36	Mechanical Principles Used in Hydraulic Dredges (1914 Partial).
756	1.30	9	Experimental Data on Coefficient of Flow for Natural River Conditions (1914 Partial).
757	14.60	94	Flood Control (1914 Partial).
762	5.70	9	Construction of Small Locks (1914 Partial).
765	2.00	22	Suction Dredges, Design and Cost of Operation (1914 Partial).
766	3.00	29	Dredges and Excavating Machinery (1914 Partial).
792	3.00	14	Quay Wall of the Schiekold, Rotterdam (1914 Partial).
815	4.70	6	Velocity of Water Necessary to Lift Broken Stone in Pipe (1914 Partial).
849	4.40	12	Work of A. R. Harlacher in River Hydraulics (1915 Partial).
870	3.60	12	Description, Cost of Erection and Operation of Suction-Dredge Plants for Sand and Gravel (1915 Partial).
879	9.90	140	Harbors and Waterway Terminal Equipment (1915 Partial).
880	1.80	23	Waterway Transportation and Port Administration (1915 Partial).
890	8.10	45	The Atchafalaya River (1915 Partial).
893	4.70	23	Silt (1915 Partial).
895	3.90	64	Freight Handling Equipment to be Used in Connection with Small Docks on Rivers and on Boats (1915 Partial).
896	0.70	4	Cost of Carrying Freight on Small Rivers (1915 Partial).
904	1.30	1	Manchester Ship Canal: Data as to Cost and Earnings (1915 Partial).
915	2.90	25	Prevention of Erosion of Sand Beaches by Ocean Surf (1915 Partial).
923	1.60	9	Tampico Harbor Works (1916 Partial).
935	2.60	18	Canal Tolls in European Countries (1916 Partial).
936	6.50	37	Operation and Maintenance of Canals in European Countries (1916 Partial).
942	5.00	12	The Effect of Dams on Flood Heights in Improved Rivers (1916 Partial).
961	21.28	264	Freight Handling at Waterway Terminals (1916 Partial).
962	8.10	22	Bottom-Dumping Buckets (1916 Partial).
972	6.80	7	A Certain Type of Dock Construction (1916 Partial).
978	3.00	4	Use of Wire Netting and Stone in Mattress Work for Shore Protection (1916 Partial).

* The original cost of the search is given in each case, and copies may be obtained at the price stated. There is a discount of 50% to members of the Society.

MEMOIRS OF DECEASED MEMBERS.

DEXTER BRACKETT, M. Am. Soc. C. E.*

DIED AUGUST 26TH, 1915.

Dexter Brackett, the only son of Cephas Henry and Louisa Thwing (Pierce) Brackett, was born on November 30th, 1851, in Newton, Mass. During his childhood his parents removed to the adjoining town of Brighton, which subsequently became a part of the City of Boston. He was educated in the town schools and was graduated from the Brighton High School in 1868. In 1868-69 he took a six months' course at a business college.

As is the case with many young men, he had no well-defined plan as to his life work, but, in March, 1869, when he was 17 years old, an opportunity was presented for his employment in the office of the City Engineer of Boston, and as engineering appeared to him to be a desirable line of work, he accepted the position. The time proved to be a favorable one, as the City Engineer's office had been recently re-organized, and the great increase in the territory of the city, by the annexation of several towns, one after another, from 1868 to 1874, caused a large increase in the engineering work to be done, with corresponding opportunities for the promotion of those who had energy and ability. Only one of the five municipalities annexed was provided with a public water supply, and Mr. Brackett was soon assigned to the work incident to the extension of the pipes into these municipalities, and the establishment of a new high-service system of supply. The great fire in Boston, in November, 1872, also called urgent attention to the necessity for a more adequate system of main pipes in the city proper.

One of the most noteworthy events in the early part of Mr. Brackett's professional career, as well as in the careers of many of his associates, was the election, in the spring of 1873, of Joseph P. Davis, M. Am. Soc. C. E., as City Engineer of Boston. His personality and the high grade of his professional work moulded to a large extent the subsequent careers of those under him. The office served as a most valuable training school, both in engineering and in ethics, a fact which was fully recognized by Mr. Brackett during his life, as well as by many others among his associates.

Under Mr. Davis' direction, Mr. Brackett was early placed in charge of the distribution system of the water-works. In addition to the large amount of work done in extending and reinforcing the water-

* Memoir prepared by Frederic P. Stearns, Past-President, Am. Soc. C. E.

pipe system in these years, the question of the waste of water soon came to the front, on account of the increasing water consumption, and, as early as 1878, Mr. Brackett began a study of this problem for Mr. Davis. At that time, although the waste of water was recognized as a very important factor, it was not nearly as large as in subsequent years. Water meters were then much more expensive and less durable than at the present time, so that the remedial measures proposed were in the line of the restriction of waste through its detection by inspection, aided by the use of Deacon meters. Church stop-cocks and Bell water-phones were also tried as aids in this work.

During the next few years, Deacon meters were set in some districts of the city, and a very marked reduction in the consumption and waste of water in those districts resulted from their use. As the result of this and other work along these lines, Mr. Brackett took especial interest in matters relating to the consumption and waste of water, and became an authority on the subject. In addition to material prepared for the official city reports, he presented papers on the subject to the New England Water Works Association in 1886* and to the American Society of Civil Engineers in 1895.†

In 1895, he was asked by the State Board of Health of Massachusetts to advise as to the present and future consumption of water per inhabitant in the Metropolitan Water District, and made a valuable report‡ on this subject. In regard to this report, it may be stated that his predictions made in 1894, as to the probable water consumption per inhabitant 30 years later, now seem likely to be verified.

In 1904, he made a very complete report to the Metropolitan Water and Sewerage Board§ on the measurement, consumption, and waste of water supplied to the Metropolitan Water District, which was transmitted to the State Legislature as a basis for legislative action in regard to the restriction of waste. This will be referred to subsequently.

During Mr. Brackett's long connection with the Boston Water-Works, which continued until August 1st, 1895, he had experience with every phase of the distribution department of a water-works. His work included, not only engineering, but practical work as Superintendent in direct charge of the constructing and operating forces, from February, 1888, to June, 1891, and covered such features as the laying of pipes across navigable streams, the raising of large mains when full of water, the cleaning of tuberculated pipe, the testing of fire engines and

* "Water Waste", *Journal*, New England Water Works Assoc., Vol. I, No. 2, p. 10 (December, 1886).

† "Consumption and Waste of Water", *Transactions*, Am. Soc. C. E., Vol. XXXIV, p. 185.

‡ Special Report, State Board of Health of Massachusetts, on a Metropolitan Water Supply, 1895, p. 157.

§ Annual Report, Metropolitan Water and Sewerage Board, January 1st, 1904, p. 301; also, *Journal*, New England Water Works Assoc., 1904, Vol. XVIII, p. 107.

fireboats, the construction of a high-pressure system of piping for fire protection only, the installation and testing of heavy pumping machinery, and the construction of distributing reservoirs.

Mr. Brackett often visited other cities to gain information as to improvements in water-works practice adopted by them, and to note the character of their works as compared with those of Boston. One of his notable studies of this kind was a comparison of the capacity of the distribution system of Boston with those of the cities of New York, Chicago, Philadelphia, Baltimore, St. Louis, Cincinnati, Cleveland, Pittsburgh, Washington, Brooklyn, and Detroit, made and reported on in 1892. It was his aim to keep the portion of the water-works under his charge up to the highest standard of practice, and in this he succeeded.

On August 1st, 1895, the engineering force of the Metropolitan Water-Works was organized, and Mr. Brackett, on account of his pre-eminent qualifications for the place, was selected as Engineer of the Distribution Department, in charge of the construction of the pipe lines, pumping stations, and reservoirs in the Metropolitan Water District. The essential parts of the works were planned and executed rapidly and well, and had advanced far enough by January 1st, 1898—the date fixed in the legislative act for the introduction of the Metropolitan water—to permit the water to be conveyed to all parts of the District on that day.

The pumping stations constructed and maintained for many years under the direction of Mr. Brackett are regarded by many who have visited them as models in respect to economy and efficiency.

On February 1st, 1907, Mr. Brackett became Chief Engineer of the Metropolitan Water-Works, and held this position until his death on August 26th, 1915. In this capacity, his duties related largely to the maintenance and operation of works already constructed, a kind of work which he enjoyed. He was very successful in selecting his assistants in charge of the various portions of the works, and, under his supervision, an extremely efficient and loyal force was developed. There were, however, during this time important additions to the works, among which may be mentioned the hydro-electric plant at the Wachusett Dam at Clinton, Mass., which has been very successful both mechanically and financially, and a similar plant at the Sudbury Dam, at Southborough, now under construction, which was designed under Mr. Brackett's direction.

One important reform took place on the Metropolitan Water-Works after Mr. Brackett was appointed Chief Engineer, namely, the introduction of water meters. This resulted largely from reports written by him on the subject of consumption and waste of water, and from his persistent advocacy of the use of meters within the Metropolitan Water District as a means of reducing waste. He had the privilege

of seeing his views adopted and the consumption of water thereby greatly decreased. Up to 1907, the consumption had been increasing so rapidly as to point out the necessity for an additional supply within the next few years, but the gradual introduction of meters reduced it so rapidly that no additional supply will be needed for many years; it is now less than two-thirds of what it would have been had the former rate of increase continued.

In addition to his regular engineering work, Mr. Brackett was called as an expert in connection with water supplies at Fall River, Stoughton, Taunton, New Bedford, Ashburnham, and Springfield, Mass.; Auburn, Me.; Syracuse, N. Y.; Atlantic City, N. J.; Louisville, Ky.; Springfield, Mo.; Harbor Springs, Mich., and other places, in several cases in connection with the testing of pumping engines or the valuation of the works.

He ceased to accept the many engagements offered him along these lines after 1895, at first because the regular work on which he was engaged demanded all his time, and later, because a diminishing vitality made it undesirable for him to accept the burden of work in addition to that required by his regular duties.

In all, Mr. Brackett served the City of Boston and the Metropolitan Water District for 46 years, most of the time in positions of much responsibility. Taking into account his entire trustworthiness, his loyalty to his work, and his special ability in the branches of water-works engineering in which he was engaged, it is difficult to overestimate the value of the public service which he rendered during his long professional career.

He had a large circle of friends, both inside and outside of the Profession, who will miss him greatly. One of his associates in the New England Water Works Association, in 1904, in a paper entitled "Some Reminiscences",* refers to him thus:

"Dexter Brackett, who assumed the presidency at this time [1889], is an ex-president who is still young, very much alive, and a familiar figure to you all, and, what is still more to the purpose, he proposes to keep up his youthful ways and indomitable energy as long as he lives. He put a tremendous amount of labor into the work of his administration. There was no lack of inspiration while his hand was at the helm. He was a working president in every sense that the word implies. His deep interest in our affairs continues unabated, and the many helpful services which he is constantly rendering are always appreciated."

It is worthy of note that at the Convention of the New England Water Works Association, held a short time after his death, there were many members of the Association who thought Mr. Brackett's

* By R. C. P. Coggeshall, *Journal*, New England Water Works Assoc., Vol. XVIII, p. 313.

name should be perpetuated by some appropriate form of memorial, and suggested the raising of a fund, the annual income of which might be used as a prize for a meritorious paper, or otherwise. A resolution was adopted by the Convention providing for the appointment of a committee to consider the subject.

Mr. Brackett was President of the Boston Society of Civil Engineers in 1897-98; President of the New England Water Works Association, 1889-90; and a member of the American Water Works Association, and the Boston City Club.

Some of the more important papers contributed by him to the technical societies, in addition to those on the subject of the consumption and waste of water already referred to, related to the raising or otherwise moving of large water mains, the consumption of water at a large fire in Boston on November 28th, 1889, the tuberculation of water pipes, the freezing of fresh water in a pipe submerged in salt water, and a description of the Metropolitan Water-Works presented to the American Water Works Association. He was an important member of the Committee of the New England Water Works Association which prepared standard specifications and drawings for cast-iron water pipe and special castings, which were adopted by the Association on September 10th, 1902.

On September 21st, 1875, Mr. Brackett was married to Miss Josephine Dame, who survives him. He also leaves one son.

Mr. Brackett was elected a Member of the American Society of Civil Engineers on June 6th, 1888. He also served as a Director of the Society from 1908 to 1910, inclusive.

WALTER FRANK CARR, M. Am. Soc. C. E.*

DIED FEBRUARY 2D, 1916.

Walter Frank Carr was born in Holyoke, Mass., on January 1st, 1861. He was the only son of Francis E. Carr, who married Sarah Kittridge, at Clinton, Mass., in 1860.

Mr. Carr's early education was received at Clinton, Mass., and he was graduated from the High School in 1878. He then entered the Massachusetts Agricultural College, at Amherst, from which he was graduated in 1881. Afterward, he entered the Massachusetts Institute of Technology, and received the Engineer's degree in 1884. His graduation thesis was on the sewerage system for his home town, Clinton. When that city took up the matter, later, it was decided to accept Mr. Carr's work as the basis for the new system, which was built and is still working successfully.

From 1884 to 1885, he was Assistant Bridge Engineer on the old Boston and Lowell Railroad, now a subsidiary of the Boston and Maine Railroad. From 1885 to 1888, he was Professor of Engineering at the University of Minnesota. In August of the latter year, he returned to railroading, as Assistant Engineer of the Minneapolis and St. Paul Street Railroad Company, and two years later was appointed Chief Engineer. In this capacity he had supervision over the electrification of 190 miles of road, including overhead and track construction, power-houses, design of rolling stock, etc.

In 1888, Mr. Carr laid out what is now the Washburn Park Addition to Minneapolis, Minn. This was one of the first attempts ever made in the United States to lay out a city on the basis of landscape architecture, and it is still considered one of the prettiest of city additions. In 1889, he laid out Horseshoe Island Park, at Eau Claire, Wis., which was also according to the "City Beautiful" idea. Both these parks are residence additions.

In December, 1891, he became General Manager and Chief Engineer of the Roanoke (Va.) Street Railway and the Roanoke Electric Light Company, and supervised the reconstruction, extension and operation of the system. It was during this year that he designed the first "Shanghai" Tee-rail now so commonly used.

Between March 1st, 1895, and June 1st, 1896, Mr. Carr was Superintendent of electric construction of the West Chicago Street Railway (Yerkes' System), covering the period of the change in motive power from horse to electricity. He also superintended the electrification of the Lake Street Elevated Railroad (Chicago and Oak Park Railroad).

* Memoir prepared by Bertram D. Dean, M. Am. Soc. C. E.

In June, 1896, he was appointed Chief Engineer of the Yerkes Company, filling that position until April, 1899.

At the time of the organization of the Falk Company, Milwaukee, Wis., steel founders and manufacturers, in 1899, Mr. Carr took a financial interest in that Company, and was appointed Chief Engineer, which position he held for twelve years. During that period he had entire management of the Frog and Switch Department, and of the contract work in the construction of city and interurban railway systems. In 1902, he accomplished one of the most difficult engineering feats of his career. In Kansas City, Mo., it was necessary to shift a two-track system from the side to the middle of the street. It was a cable-power road traversing the down-town district, and the entire work was accomplished, without interrupting regular service in the slightest degree. In 1909, he designed and patented the first three-way switch ever constructed.

During the last four years, Mr. Carr was on the Pacific Coast, for two years as Manager of the Railway Department of Parrott and Company, with headquarters in San Francisco; later, as Pacific Northwest Manager of the Western Engineering Sales Company, with headquarters in Seattle. He also practised as a Consulting Engineer, specializing in valuation of railways and public utilities.

At the time of his death, Mr. Carr was a member of the National Geographic Society, the Engineers' Club of San Francisco, Engineers' Club of Seattle, Railway Signal Association, Technology Club of Seattle, Municipal League, Eureka Lodge No. 20, F. and A. M., and other organizations.

He was married in 1885, to Alice Merril, at Mansfield, Mass. Mrs. Carr died two years later. On May 28th, 1890, he married Elizabeth Gardner, of Minneapolis, Minn., who, with three sons, Gardner, Alan, and Burton, survives him.

Mr. Carr was a man of high ideals with a keen conception of his profession. Strength and clearness formed the basis of his daily work; his standard was that of honesty and truth. His modest, lovable disposition endeared him to all. To his many friends, his passing away is a source of great sorrow, but they have the consolation of knowing that it had been their privilege to work and walk with so interesting a man.

Mr. Carr was elected a Member of the American Society of Civil Engineers on June 6th, 1894.

LOOMIS EATON CHAPIN, M. Am. Soc. C. E.*

DIED JUNE 18TH, 1915.

Loomis Eaton Chapin, son of Zerah Chapin and Lucy (Starkweather) Chapin, was born on May 7th, 1858, in Weathersfield, Ill. His father was a miller operating what in those days was considered an extensive flour mill. During his childhood, his parents removed to Kewanee, Ill., and there he received his early education in the public schools and high school. Later, he entered the University of Michigan, and was graduated in the Class of 1883 in the Department of Civil Engineering.

As is frequently true in the early experience of many engineers, Mr. Chapin worked during the school vacations in various capacities and on miscellaneous engineering work. Thus, he was employed on the surveys of the Toledo, Ann Arbor, and Grand Trunk Railway; on the Wheeling and Lake Erie Railroad Bridge over the Maumee River; later, by the Wheeling and Lake Erie Bridge Company on similar work at Toledo; and then by the Toledo Belt Railway Company on surveys.

On graduation, in May, 1883, Mr. Chapin entered the employ of the City of Toledo, Ohio, as Assistant Engineer, which position he held until 1887. During this time he had charge of the construction of many important public works and street improvements, among which was the Cherry Street Bridge over the Maumee River.

From 1887 to September, 1891, he was Associate Engineer with J. D. Cook, Hydraulic Engineer, and was engaged on the design and superintendence of water-works and sewerage systems in many cities and towns throughout Ohio, Indiana, and Michigan. The experience and development obtained under Mr. Cook, who was an old-fashioned, dignified, courteous engineer, doing important work at that time, was excellent for the young men who had opportunity to be in his office. Undoubtedly, the excellent qualities of Mr. Chapin's later life reflected the training thus obtained. In September, 1891, Mr. Chapin became City Engineer of Canton, Ohio, and served during the progressive growth of this municipality, when many important engineering problems were solved. Among these, that which stands out pre-eminently was the construction of the sewage treatment works in which chemical precipitation was utilized. This was at a time when the treatment of sewage was just coming into vogue, and the necessity of treating these obnoxious human wastes, in order that nuisances might be avoided and the spread of infectious disease prevented, was just beginning

* Memoir prepared by Morris Knowles, M. Am. Soc. C. E.

to be realized. The chemical precipitation works, following closely after the completion of the similar work at Worcester, Mass., were based on the investigations and report of Samuel M. Gray, M. Am. Soc. C. E., Consulting Engineer, of Providence, R. I., and were built under Mr. Chapin's supervision. This experience gave him an early insight into the problems of sewage disposal, and caused him to be called upon as a consultant in many similar problems in other cities in the Middle West.

At about this time, he engaged in consulting engineering practice, with an office in Canton, Ohio. From 1902 to 1907, he was associated with Morris Knowles, M. Am. Soc. C. E. During this period an office in Pittsburgh, Pa., was opened, and, after 1907, this, as well as the office in Canton, Ohio, was continued by Mr. Chapin to the time of his death. During this time, many water-works systems, purification plants, sewerage systems, and sewage treatment plants were designed and erected under Mr. Chapin's direction, and his reports on municipal improvements are to be found in the files of many cities and towns extending from the Allegheny Mountains throughout the Middle States.

Later, Mr. Chapin paid a great deal of attention to water-works valuation, particularly referring to rate cases, for clients many of whom he had previously served in other ways. He also built several industrial and manufacturing water supplies for corporations and groups of mining and operating companies, and was the regular advisor of many of his clients. At the time of his death he was continuous Consulting Engineer for the Beaver Valley Water Company, the North Michigan Water Company, and several other companies, as well as some municipalities.

Mr. Chapin was of a genial temperament and was a companion full of fellowship. He was an indefatigable worker, even to the exhaustion of his own strength and resources, but ever had the sunny point of view, and was universally liked by clients and brother engineers.

He was married on October 13th, 1886, to Elizabeth Hadley, of Ann Arbor, Mich., who survives him, together with one son, Robert Hadley, and two daughters, Persis Louise and Elizabeth Loomis.

Mr. Chapin was elected a Junior of the American Society of Civil Engineers on December 3d, 1884; an Associate Member on September 7th, 1892; and a Member on November 4th, 1896.

AMORY COFFIN, M. Am. Soc. C. E.*

DIED JUNE 5TH, 1916.

Amory Coffin, the eldest son of Dr. T. C. Amory Coffin and Jessie M. (Edmondston) Coffin, was born in Charleston, S. C., on August 9th, 1841. He received his early education at a military school in Yorkville, S. C., and, later, entered the South Carolina Military Academy, known as the "Citadel", at Charleston, from which he was graduated in 1862.

Mr. Coffin was in command of the squad which fired what was practically the first gun of the Civil War, namely, an alarm gun to notify the batteries around Charleston that the U. S. Steamer *Star of the West* had been sighted farther up the coast, bound for the relief of Fort Sumter.

During the war he was engaged most of the time in and around Charleston with a battery known as the Washington Artillery. In one of the larger skirmishes, while acting as Adjutant, he was severely wounded in the head and disabled for several months.

After the close of the war Mr. Coffin secured such employment as was available in that unhappy region, teaching school, surveying, and, with his father, even translating into English the German novel, "Merchant of Berlin", by Mühlbach.

In 1867 he was engaged by the Phœnix Iron Company, at Phœnixville, Pa., in making designs and estimates on iron structures. He made a survey of the plant of the Company and of a large part of the Town of Phœnixville, and designed most of the mills and shops there. Mr. Coffin edited the handbook of the Company, which was the first of its kind and has been so generally copied by all makers of structural shapes. The graphical table showing the values of beams for various spans and floor loads was originated and copyrighted by him in 1885.

He had an exceedingly fine sense of the proper proportion in all things, and was most painstaking to secure the best results. His designs for the rolls with which the Company made the various sections remained unchanged until the Standard Sections, now in general use, were adopted, and from which they differ but slightly.

In November, 1885, Mr. Coffin became Chief Engineer of the Phœnix Iron Company and remained in that capacity until March, 1896. He designed the structural features of many of the notable buildings of that period, among them being Madison Square Garden, New York, the Crocker Building, San Francisco, which was provided with earthquake bracing, the Provident Life and Trust Company Building, Philadelphia, the Equitable Building, Baltimore, and numer-

* Memoir prepared by T. Amory Coffin, M. Am. Soc. C. E.

ous buildings for the Government in Washington in connection with the Supervising Architect's office.

In 1896, Mr. Coffin moved to New York City to take charge of the engineering work of the well-known architect, the late George B. Post, M. Am. Soc., C. E., with whom he was associated until 1909.

During this period many buildings of considerable importance came under Mr. Coffin's supervision, among them being the St. Paul Building, New York, and the New York Stock Exchange, where many exacting conditions of structure and foundation were successfully met.

He prepared the engineering designs for the College of the City of New York, the Prudential Life Insurance Company buildings and the Mutual Benefit Building in Newark, N. J., the Vincent Building, New York City, and the Wisconsin State Capitol, at Madison. In the design of the steelwork for the Williamson Building, Cleveland, Ohio, he utilized the features of the then popular Z-bar column to economize on the floor system and column design by allowing the girders to pass through the columns on their central axes, thus securing the benefits of cantilever construction and central column loading.

Mr. Coffin also was Consulting Engineer for Messrs. Phelps, Dodge and Company on the bridge at El Paso, Tex., and for several other bridges on the line of the El Paso and Southwestern Railway.

He was of a quiet and retiring disposition, and spent most of his time with his family and his friends. Those who knew him, either in a business or social way, always found him affable and kind; for him it was never any trouble, but rather a delight, to give the fullest explanation possible on any subject, and his friends often availed themselves of his great store of information, gathered by a life of study and of experience and put to the fullest possible use. For several years Mr. Coffin was President of the Board of Health of Phœnixville, and a Trustee of the Phœnixville Library.

On January 14th, 1868, he was married to Emma Hopkinson, of Edisto Island, S. C. In 1912, he returned to Phœnixville where he resided until the death of his wife, on February 15th, 1916. On account of his failing health, it became necessary to seek a more suitable climate, and in March of that year he moved to the home of his son, Francis H. Coffin, at Scranton, Pa., where, after several recoveries and relapses, he finally passed away on June 5th, 1916.

He is survived by three sons, Thomas Amory Coffin, M. Am. Soc. C. E., Francis Hopkinson Coffin, and Laurence Edmondston Coffin.

Mr. Coffin was elected a Member of the American Society of Civil Engineers on March 3d, 1875.

WILLIAM WEEDEN COLE, M. Am. Soc. C. E.*

DIED DECEMBER 20TH, 1915.

William Weeden Cole was born on April 4th, 1866, in Medford, Mass., of Revolutionary stock. He acquired his technical education at the Worcester Polytechnic Institute, with the Class of 1887. After leaving college, he was employed as Assistant Engineer with the New England Construction Company, in charge of water-works construction and operation, at Newport, R. I., Gloucester, Mass., Frankfort, Ind., and Green Bay, Wis. He was subsequently with the Toledo, St. Louis and Kansas City Railroad, as Division Engineer, but, owing to an injury received in this service, returned to Boston and entered the employ of the Thompson, Houston Company, with whom, after three months' factory service, he became Superintendent of Electrical Affairs on the Alston Division of the West End Street Railway.

In 1892 Mr. Cole became General Manager for the Utica Belt Line Railroad, and a year later designed and built the West Side Street Railway and the Elmira Heights Water-Works, at Elmira, N. Y. During the next sixteen years he was associated with the Public Utility Companies in Elmira, being instrumental in consolidating the street railways, electric light companies, natural and artificial gas companies, water-works companies, and park properties, into one consolidated Utility Corporation, and served as Vice-President and General Manager of these properties. During a large part of this period, he also acted as Consulting Engineer for the Chemung Canal Trust Company, of Elmira, in connection with its various investments.

In 1907 Mr. Cole left Elmira to become a member of the firm of Day and Zimmerman, of Philadelphia, Pa., well known in engineering and public utility circles. He acted as Manager of their Public Utility Department, and remained with them in that capacity for seven years. The organization of the Pennsylvania Central Heat, Light and Power Company, which includes a number of electric light and gas properties, also the reorganization and reconstruction of the Electric Light and Traction Companies of Oil City and Franklin, Pa., were effected by Day and Zimmerman during Mr. Cole's management of their Public Utility Department.

Returning to New York, Mr. Cole, in January, 1915, entered into a partnership with Messrs. Arthur S. Ives and Rolland A. Davidson, who had previously been associated in engineering work, and the new firm of Cole, Ives and Davidson has continued to specialize in public utility engineering work and in financial reports for prominent banking interests.

* Memoir prepared by Arthur S. Ives, Esq.

In addition to his other activities Mr. Cole acted at different times as Consulting Engineer for various financial interests in connection with appraisals, examinations, and reports on gas, electric, and traction properties, including the utilities at Canandaigua, N. Y., Hartford, Conn., Lancaster, Pa., Colorado Springs, Colo., Lockport, N. Y., Topeka, Kans., Terre Haute, Ind., Denver, Colo., Baltimore, Md., Beloit, Wis., etc.

Mr. Cole was also the first President of the Empire State Gas and Electric Association, of New York, and Vice-President of the New York Street Railway Association. He was a member of the American Society of Electrical Engineers, National Electric Light Association, Illuminating Engineering Society, American Water Works Association, Engineers' Club of New York, Racquet Club of Philadelphia, Elmira City Club, Venango Club of Oil City, Railroad Club of New York, and others. He was also a thirty-second degree Mason.

Mr. Cole had a most winning personality and a host of friends. He had a persuasive manner which, both in business and in his social life, kept him surrounded by those who could not help but be attracted by the exquisite wit and story-telling ability which he so largely possessed. He was a brilliant public speaker and possessed of a most remarkable memory; he had a marvelous faculty of quoting statistics, a constant source of wonderment to his engineering friends, and with a corresponding effect on his clients and laymen with whom he came in contact.

Mr. Cole had also a faculty which excited the greatest comment, in that, when he was making an engineering examination of a property, he was never known to make notes or memoranda, yet his reports were replete with data that could only be secured by personal observation and held by his retentive memory. He seemed to have an intuitive judgment in such matters, which was so invariably correct as to win and preserve the confidence of the large financial interests for whom he did so much of this work.

He was married on June 14th, 1889, to Miss Agnes Cornelia Barker, of North Granville, N. Y., who, with two sons, Norman Weeden Cole and Lester Weeden Cole, survives him.

Mr. Cole was elected a Member of the American Society of Civil Engineers on October 7th, 1903.

CLOUD CLIFFORD CONKLING, M. Am. Soc. C. E.*

DIED MAY 8TH, 1916.

Cloud Clifford Conkling was born in Rensselaerville, N. Y., on October 6th, 1862. He spent his boyhood in Scranton, Pa., where he attended the public schools.

While continuing his studies, including Civil and Mining Engineering, he obtained employment, in September, 1881, as Rodman on the Erie and Wyoming Railroad, and held a similar position, during the summer of 1882, on a water supply survey of the Ramapo Valley, N. Y.

In 1882 and 1883, Mr. Conkling took a special course of study at Williston Seminary, East Hampton, Mass.

During 1884 and 1885, he was engaged as Draftsman and Transitman in the office of an engineering firm at Scranton, Pa., and on a railroad survey in New Jersey under Mr. J. H. Rittenhouse.

In September, 1886, Mr. Conkling entered the service of the Lackawanna Iron and Steel Company (now the Lackawanna Steel Company) as Rodman in the Civil and Mining Engineering Department. He served in various positions with this Company until 1904, as follows: April, 1888, as Transitman; September, 1892, as Assistant Engineer; and February, 1894, as Chief Civil Engineer, in which capacity he had charge of the design and construction of all the civil engineering work of the Company, including the construction of its plant at Buffalo, N. Y., as well as the design and construction of all foundation work, railroad yards, sewers, villages, etc., in connection with it. As Chief Engineer of the South Buffalo Railroad, he had charge also of the design and construction of all canal and harbor works connected with that railroad.

In 1904, Mr. Conkling left the direct employ of the Lackawanna Steel Company to engage in private practice as a Consulting Engineer, in Buffalo, N. Y. In this capacity he made the surveys, plans, and estimates for the Buffalo, Depew and Lake Erie Railroad, a terminal freight line built around the City of Buffalo, and was engaged on other important work.

In 1907, the original design of the Lackawanna Steel Sheet-Piling was patented. Mr. Conkling designed and patented modifications of the original design, and was instrumental in bringing to a successful completion the designs and sections of the steel sheet-piling now rolled by the Lackawanna Steel Company. He also advised with engineers on many important projects in which this piling was used.

In October, 1913, the consulting work of this character had reached such proportions that Mr. Conkling returned to the employ of the

* Memoir prepared by Charles S. Boardman, M. Am. Soc. C. E.

Company, as Chief Engineer of the Steel Sheet-Piling Department, which position he held at the time of his death on May 8th, 1916.

Mr. Conkling's long service with the Lackawanna Steel Company, his faithfulness to duty and fairness to his associates, made for him many friends, and his death will be a severe loss, not only to his Company and friends, but also to the Engineering Profession.

He had served for 12 years in the National Guard of the State of Pennsylvania, retiring as a Sergeant-Major of the Thirteenth Regiment.

Mr. Conkling was elected a Member of the American Society of Civil Engineers on January 4th, 1905.

HERBERT WHEELER COWAN, M. Am. Soc. C. E.*

DIED MAY 29TH, 1915.

Herbert Wheeler Cowan was born in Bath, Me., on April 27th, 1862. He was the son of Augustus Nelson Cowan and Martha Jane (Chapman) Cowan, and was a descendant of the fifth generation of James Cowan, of Scotch ancestry, who came from the North of Ireland and settled in Merrimac, N. H., about 1750. His great-grandfather served for three years as a soldier in the War of the Revolution. His father was a ship-builder and store-keeper in Bath.

After a preliminary common school education in his native town, Mr. Cowan entered Worcester Polytechnic Institute, Worcester, Mass., in 1879, and was graduated therefrom with the degree of Bachelor of Science in 1882. The somewhat meager records of the Institute indicate that he was an excellent student.

From June, 1882, to January, 1883, Mr. Cowan was employed as Draftsman by Fuller and Delano, Architects, of Worcester, Mass. On March 6th, 1883, he went to St. Paul, Minn., where he joined his classmate, Mr. Fred W. Moore, and the two young engineers then proceeded as far as Winnipeg, Man., arriving there on March 29th. On April 26th, they secured employment with a contractor as Chainmen with a surveying party which proceeded westward by rail to Swift Current, then the western terminus of the Canadian Pacific Railway; thence the journey was continued on foot 450 miles to the Saskatchewan River country in the neighborhood of Edmonton, Alberta. This party was engaged in the subdivision of townships which had been laid out previously by the Canadian Government. After spending the entire summer in this region and enduring the many hardships incident to frontier surveying work, Messrs. Cowan and Moore decided that they were wasting their time, and so, with seven others, they left camp early in September, walking some 200 miles south to Calgary, Alberta, which place had been reached during the summer by the Canadian Pacific Railway.

Arriving in St. Paul, Minn., on September 12th, 1883, Mr. Cowan held several temporary positions of minor importance. However, on December 10th, his career as a railroad engineer may be said to have begun, for, on that date, at Prairie du Chien, Wis., he entered the employ of the Winona, Alma and Northern Railroad Company, as Topographer, Instrumentman, and Draftsman on a preliminary survey of its line along the east bank of the Mississippi River. This line was acquired later by the Chicago, Burlington and Northern Railroad Company, and has become a part of what is now the main line

* Memoir prepared by Herbert S. Crocker, M. Am. Soc. C. E.

of the Chicago, Burlington and Quincy Railroad between Chicago and St. Paul.

In December, 1884, Mr. Cowan was transferred to La Crosse, Wis., where he became Draftsman in the office of the Chief Engineer, the late William B. Lawson, M. Am. Soc. C. E. He remained there until July, 1886, when he was transferred by Mr. Lawson to Colorado, where he was employed as Topographer on the then contemplated extension west, from Denver to Salt Lake City, of the lines of the Chicago, Burlington and Quincy Railroad Company. In this field work, he was a member of a party, headed by Mr. T. R. Countryman, which conducted the survey between Denver and the summit of the main range of the Rocky Mountains. This work lasted until February, 1887, when, after two months' service in the office, Mr. Cowan was transferred to field work west of Salt Lake City for the same company and, until December, 1887, was engaged in reconnoitering—for the most part on horseback—a projected line to Los Angeles, Cal. He then returned to office work in Denver, which work continued until July 1st, 1888, when he received word to report, at Alliance, Nebr., to Maj. R. J. McClure, the Consulting Engineer of the Company, who made the first explorations of a railway line to extend the Burlington System to the Northwest, this being an initial step toward what was later a connection with the railways reaching to the Pacific at Puget Sound and into British Columbia. Mr. Cowan's selection for this reconnaissance was due to his splendid record on previous work of this character. His journey with Maj. McClure was made by team consisting of bronchos and buckboard, by which method of travel they covered 1 400 miles in 60 days, extending their trip through Yellowstone Park and returning to Alliance in August, 1888, at which time Mr. Cowan's connection with the Chicago, Burlington and Quincy Railroad Company ceased because of his desire to locate permanently in Colorado.

From September 1st, 1888, to April 1st, 1890, Mr. Cowan was an Assistant Engineer on the Colorado Division of the Union Pacific Railway, and from April 1st, 1890, to December 18th, 1893, he served in a similar capacity for the Union Pacific, Denver and Gulf Railway Company, a subsidiary of the Union Pacific Railway Company, in both of which positions he was engaged on construction, maintenance of way, and office work, and reported to the Division Engineer, William Ashton, M. Am. Soc. C. E.

In 1893, the Union Pacific System went into the hands of Receivers, separate Receivers being appointed for many of the subsidiary lines, and, in February, 1894, Mr. Cowan was appointed Resident Engineer for the Receiver of the Union Pacific, Denver and Gulf, and for the Denver, Leadville and Gunnison Railway Companies. He held these positions throughout the receiverships of these roads and until the

reorganization of the properties, in January, 1899, into what is now known as the Colorado and Southern Railway Company. Mr. Cowan then became Chief Engineer of this company and retained that position until his death.

Of the more important work done by Mr. Cowan since 1893, mention may be made of the following:

In 1895 he had charge of the surveys and construction of approximately 27.5 miles of railway between Walsenburg and Trinidad, Colo., connecting various mine branches to form a main line between the points mentioned.

During 1900 he had charge of the plans and construction of the Colorado and Southern Railway Company's roundhouse, shops, and other terminal facilities in Denver, the headquarters of the System. In 1902, he acted as Consulting Engineer for the Fort Worth and Denver City Railway Company, in connection with the construction of the principal shops and roundhouse, as well as the yards, at Childress, Tex. During the next few years he served that company in a similar capacity in connection with the construction of terminals at Amarillo, Tex.

During 1902 and 1903, Mr. Cowan had charge of the preliminary and location surveys and the construction of 11 miles of standard-gauge line built from Fort Collins to Wellington, Colo., through an agricultural section which, at that time, was just being brought under cultivation. In 1905 and 1906 this construction was continued northward in several branches from Fort Collins, some 26 miles in all.

Also, during 1904, 1905, and 1906, Mr. Cowan acted as Consulting Engineer in connection with the surveys and construction of approximately 300 miles of standard-gauge line for the Trinity and Brazos Valley Railway Company, between Cleburne and Teague, Tex., and also between Waxahachie and Houston in the same State. This line had the joint financial support of the Colorado and Southern Railway Company and the Chicago, Rock Island and Pacific Railway Company, and was constructed under the joint direction of their Engineering Departments.

From 1906 to 1908, inclusive, he located and constructed approximately 17 miles of second track paralleling the main line of the Colorado and Southern Railway between the northern limits of Denver and extending northwestward to Louisville Junction. This work involved a reduction of gradient from 1.25 to 0.8 per cent. A portion of this road is operated by the Denver and Interurban Railroad Company as an electric line.

From 1908 to 1910, Mr. Cowan served as Consulting Engineer in the construction of 83 miles of standard-gauge line known as the Stamford and Northwestern Railway, extending northwestward from Stamford to Spur, Tex.

For some years prior to 1910, the increasingly heavy traffic on the 46 miles of main line of the Denver and Rio Grande Railroad Company between Minnequa and Walsenburg, Colo., which had been operated jointly by that company and the Colorado and Southern Railway Company, rendered it necessary to take steps to relieve the congestion. Accordingly, under the joint supervision of Mr. Cowan and Mr. J. G. Gwyn, Chief Engineer of the Denver and Rio Grande Railroad Company, surveys were made, which resulted in the construction, during 1910 and 1911, of a new line providing a double track of lighter curvature and much lower gradient. This at the present time is one of the heaviest pieces of railroad construction in Colorado; the line has been operated with great satisfaction to both the companies interested.

During 1910 and 1911 there was also constructed, under the direction of Mr. Cowan as Chief Engineer, a standard-gauge line, having a length of 32 miles, between Dixon, Colo., and Cheyenne, Wyo.

From 1908 to the time of his death, as a member of engineering boards directing their construction, Mr. Cowan was actively interested in a number of local improvements in Denver, among which were the Twentieth Street Viaduct, the West Alameda Avenue Subway, and the Colfax-Larimer Viaduct, in all of which public improvements his company, with other corporations, was interested financially.

Owing to his marked ability in handling joint work satisfactorily, together with his profound knowledge of the local railway and terminal situation, Mr. Cowan was chosen in 1912 as Chairman of an Engineers' Committee, composed of the Chief Engineers of all the holding companies constituting the Denver Union Terminal Railway Company. This Committee had charge of all the engineering work pertaining to the remodeling of the Denver Union Station, and the construction of its trainsheds, together with the re-arrangement of the passenger and terminal yards, and incidental work in connection therewith. This work involved the harmonizing of the different interests, as well as the handling of intricate details, and was only half completed at the time of his death.

In his professional life, Mr. Cowan was noted for the thorough manner in which he handled his work and for his retentive memory in connection with its details. He did not confine his attention to the duties of his own department, for, owing to his good judgment, he was constantly called into council with the executive heads of his company relative to general policies and administration. He took a great interest in the development of the resources of the country tributary to his road, and was instrumental in the projection and construction of many miles of spurs to stone quarries and coal and metalliferous mines.

On account of his honesty, good judgment, and breadth of view, his services were often sought, both in consultation relative to new projects and in the arbitration of differences of opinion.

At the time of his death, he was a Director of the Colorado Railroad Company and of the Denver and Interurban Railroad Company, both subsidiaries of the Colorado and Southern Railway Company.

On July 20th, 1893, Mr. Cowan was married to Sarah A. McMillan, who died on April 15th, 1897. On June 30th, 1898, he was married to Mrs. Clara (Lewis) Waterman, by whom he is survived. The surviving children of this marriage are Ralph Dudley and Lester Augustus Cowan.

A description of Mr. Cowan's life would be incomplete without mention of the many excellent qualities which won him the love and respect of all with whom he came intimately in contact. He was generous, fair-minded, and unselfish in securing advancement of his employees and members of his Profession. Being charitable in his judgment and opinion of others, it is not strange that, although he insisted on work of first quality, he did not arouse the enmity of those whom at times he found it necessary to oppose. Although retiring in disposition, he had many intimate friends, not only in Colorado, but throughout the country, all of whom were greatly shocked at his taking off in the prime of life and at the time when his usefulness in his community and among his circle of friends was ever increasing.

In August, 1900, Mr. Cowan became a Charter Member of the American Railway Engineering Association, the annual conventions of which he attended with regularity and, from time to time, he contributed valuable information to its Standing Committees.

On April 1st, 1914, he became a Corporate Member of the American Wood Preservers' Association, in which he took a very active interest. That Association, at a recent convention, passed resolutions of regret on his death.

Formal resolutions extolling the life and services of Mr. Cowan were adopted by the Board of Commissioners of the City of Fort Collins, Colo., and by the Board of Directors of the Colorado and Southern Railway Company.

Mr. Cowan was elected a Member of the American Society of Civil Engineers on June 3d, 1908, and, in 1909, he was one of the local members active in the formation of the Colorado Association of its members.

FOSTER CROWELL, M. Am. Soc. C. E.*

DIED MARCH 29TH, 1915.

Foster Crowell was born at Westchester, Pa., on October 13th, 1848. His ancestry traces back for more than 200 years to the earlier immigration to America. His great-grandfather was an early American sea captain, his grandfather, a schoolmaster in Philadelphia, and his father, the Rev. John Crowell, was an able and much beloved Presbyterian minister, long the pastor of the historic Brick Church, of East Orange, N. J. His mother was Katharine Roney, of North of Ireland ancestry, whose father was an American soldier in the War of 1812.

Foster Crowell's school education began in his grandfather's school in Philadelphia, and was continued in a private school in Orange, N. J. In 1863, he entered the Pennsylvania Polytechnic College, from which he was graduated in 1867 with the degree of B. C. E. Three years later the degree of M. C. E. was conferred on him by the same institution. Before entering college, at the early age of 14 (1862), he enlisted in the Home Guard of New Jersey, and saw actual military service in the draft riots of that time.

Mr. Crowell began his professional career in 1867 as a Chainman on the engineering work for the improvement of Fairmount Park, Philadelphia. He was advanced to the position of Assistant Engineer in 1868, and was in charge of the construction work, with the title of Superintending Engineer, from 1870 to 1872.

In March, 1872, he was appointed one of the two civil engineers (the other being the late A. G. Menocal, M. Am. Soc. C. E.), with the United States Government expedition, under the auspices of the Navy Department, to make examinations and surveys for the Nicaragua Canal. In the capacity of Principal Assistant Engineer, he accompanied two expeditions to that then almost unknown tropical wilderness.

In the preparatory work for the Centennial Exposition, in Philadelphia (1874-75), Mr. Crowell was Principal Assistant Engineer on the Exposition Grounds and Engineer on the ironwork in the construction of Memorial Hall and Horticultural Hall. Resigning in June, 1875, he accepted a position as Manager and Engineer of the Philadelphia Branch of the Cornell Iron Works, having charge, in this connection, of the work done by that Company on the elevated railroad construction then in progress in New York City.

In 1878, he opened an office in New York City for general professional practice. In 1880 and 1881, he was Chief Engineer of the Elizabeth City and Norfolk Railroad during its location and construction. Entering the service of the Pennsylvania Railroad Company in

* Memoir prepared by S. Whinery, M. Am. Soc. C. E.

1881, as Engineer on Construction, he had charge, for about 7 years, of the many important improvements made by the Company in that period. Among these were the building of the Philadelphia, Germantown, and Chestnut Hill Railroad, portions of the Pennsylvania and Schuylkill Valley Railroad, the Bedford and Bridgeport Branch, the new line from Cincinnati, Ohio, to Richmond, Ind., besides reconstruction work on the main line, additional trackage, and miscellaneous work.

From 1888 to his death, Mr. Crowell was engaged in practice as a Consulting Engineer in New York City. Among a large number of professional engagements during that period, as disclosed by official reports left on file in his office, the following may be noted:

Reports on the engineering treatment of the water-front of Toronto, Ont., Canada; on the Ravine du Sud, Haiti, for the Republic of Haiti; on the Delaware, Lake Erie and Western Railroad; on the Boston and Maine Railroad; on the Genesee Valley Railroad; on terminal docks at Charleston, S. C., for the South Carolina and Georgia Railroad; on the Buffalo and Susquehanna Railroad; on the Harlem River Driveway; on the condition of various railroad properties and on the filling in of the Hackensack Meadows for a prominent New York banking firm; on a projected railroad from Sacramento to Lake Tahoe, Cal.; and (in conjunction with the late George S. Morison, Past-President, Am. Soc. C. E.) on additional track facilities through Bergen Hill, N. J., for the Erie Railroad. During this period he designed and built (1900-01) the notable all-steel Redridge Dam, at Houghton, Mich.

Mr. Crowell took an active part in that body of prominent engineers who worked up and prepared the monumental report to the Merchants Association of New York on a future water supply for that City, himself preparing the Special Report (1900) on a supplementary salt-water supply for fire protection. He was an active and useful member of the Committee appointed by the same Association which, in 1903, reported on passenger transportation in the City of New York.

In 1897, he was offered, but declined, the position of Chief Engineer of the Nicaragua Canal Commission (the Walker Commission), and, in 1902, he declined the position of Chief Engineer of the Department of Bridges of New York City.

In 1907, he accepted the position of Commissioner of Street Cleaning of the City of New York, but apparently did not find the work congenial and resigned in 1909. Early in 1912, he was appointed Consulting Engineer to the President of the Borough of Queens, City of New York, serving in that capacity until his death.

Mr. Crowell became a member of the Institution of Civil Engineers of Great Britain in May, 1893. He was also a member of the Philadelphia Engineers' Club from 1883 to 1893, and was one of the organ-

izers, and a member of the first Board of Directors, of the Cincinnati Engineers' Club, while engaged on railroad work in that vicinity.

During all his life, Mr. Crowell took a keen and active interest in upholding and advancing the Engineering Profession, and contributed liberally to its literature.

To the American Society of Civil Engineers he contributed three important papers, and its *Transactions* bear further witness to his wide interest in varied branches of the Profession, his name appearing in the discussion of about fifty papers other than his own. His name also appears as the translator from the Spanish of three papers read at the International Engineering Congress of the Columbian Exposition, Chicago, 1893, and from the French of two papers read at the International Engineering Congress, St. Louis, 1904. The *Transactions* of the Engineers' Club of Philadelphia contain four papers by Mr. Crowell. Nor was his literary work confined to the professional societies; for the *Engineering Magazine*, he prepared the following articles: "Railroad Facilities of Suburban New York" (1896); "Modern Wharf Improvement and Harbor Facilities" (1896); "How Holland was Made" (1895); and for *Scribner's Magazine*, "A Little Journey to Haytii" (1891).

In 1904, he delivered the address to the graduating class of Rensselaer Polytechnic Institute.

During his busy and useful professional life, Mr. Crowell did not neglect the civic and other duties and responsibilities of the good citizen. In Flushing, N. Y., his home since 1890, and where he died on March 29th, 1915, he took an active interest, and was an influential factor, in local industrial, civic, charitable, and social affairs.

He was a member, and President, for four years, of the Flushing Association, the leading local organization in originating and promoting civic improvements, and served as a Trustee of the Flushing Hospital, and as a member of the local Board of Health, for the same number of years. He was also a Trustee of the Flushing Library Association.

Mr. Crowell was married to Anna McKinstry Whiting, daughter of John Nicholas Whiting, of Orange, N. J., on January 27th, 1881. Mrs. Crowell and two sons, the Rev. John Whiting Crowell and Francis Stirling Crowell, Assoc. M. Am. Soc. C. E., are living. Another son, Thomas Sutherland Crowell, is deceased.

For those who had the pleasure of a more or less intimate acquaintance with Mr. Crowell, it is unnecessary to speak of his personal and professional traits of character. For others, they may be briefly characterized: A man of unquestioned integrity and honor; of high scholarly and professional attainments; of courteous, but quiet and dignified bearing; a loyal friend; broad and liberal minded, but safely conservative; a man of positive convictions; a careful, clear thinker, his con-

clusions were formed with deliberation, stated with confidence, and defended with vigor; devoted and loyal to his profession, and able, painstaking, and thorough in his professional work.

He will be remembered as an exemplary man, a useful citizen, and an honor to the Profession.

Mr. Crowell was elected a Member of the American Society of Civil Engineers on December 1st, 1880. He was a member of the Board of Direction for two years (1893-95), and was also a member of the Society's Special Committee on Standard Sections of Railroad Rails, whose report, in 1893, marked an era in the design of railroad rails, and has had a controlling influence with both engineers and American railways ever since.

HENRY CLAY DERRICK, M. Am. Soc. C. E.*

DIED MAY 9TH, 1915.

Henry Clay Derrick was born on January 13th, 1832, at Washington, D. C., where his father was in the service of the Federal Government under which he held many important positions, among them being that of Chief Clerk of the State Department during the Buchanan Administration. He was a personal friend of Henry Clay, for whom his son, Henry Clay Derrick, was named.

Henry Clay Derrick was graduated from Bolmar's Academy, West Chester, Pa., and began his engineering career, in 1851, as Resident Engineer, under the late Captain Andrew Talcott, on the construction of the Richmond and Danville Railroad. In 1853, he was appointed Resident Engineer on the Pittsburgh, Maysville, and Cincinnati Railroad, where he remained until 1855.

From 1855 to 1858, he served as United States Deputy Surveyor for the Territories of Kansas and Nebraska. In 1858, he was appointed Assistant Engineer at the United States Arsenal at Harper's Ferry, Va., and took part in the stirring events of October, 1859, when John Brown captured and, for a few hours, held the Arsenal.

When the Civil War broke out in 1861, Mr. Derrick joined the Confederate Army and rendered active and conspicuous service as Captain of Engineers in the Army of Northern Virginia, until the close of the war, when he settled in Halifax County, Virginia, and resumed the practice of his profession. From 1868 to 1874 he was engaged as follows: 1868 to 1869, Division Engineer of the Norfolk and Great Western Railroad; 1869 to 1870, Resident Engineer on the Western North Carolina Railroad; 1870 to 1871, Principal Assistant Engineer on the Richmond, Fredericksburg, and Potomac Railroad; 1871 to 1872, Principal Assistant Engineer on the Piedmont and Potomac Railroad, and 1872 to 1874, Engineer in Charge on the Washington, Cincinnati, and St. Louis Railroad.

In 1875, Colonel Derrick accepted the appointment of Lieutenant-Colonel of Engineers on the General Staff of the Egyptian Army, and served with distinction for three years. During that time, he surveyed the military railroad from Massowah, on the Red Sea, to Bahr Reza, in Abyssinia. He also took part in the disastrous campaign against King John of Abyssinia, during which the Egyptian Army was almost destroyed as a fighting force, the remnant being saved by the exertions of a small group of American officers, of whom Colonel Derrick was

* Memoir prepared by the Secretary from information furnished by H. G. Prout and G. H. Derrick, Members, Am. Soc. C. E.

one. He was afterward decorated, for gallant service in the field, by the Khedive Ismail, with the Turkish Imperial Order of Medjidieh.

In 1878, Colonel Derrick returned to the United States and resumed his engineering practice, being engaged as follows: 1878 to 1879, Chief Engineer of the Danville and New River Railroad; 1880 to 1881, Resident Engineer on the Chesapeake and Ohio Railroad; 1882 to 1884, Engineer in Charge of the Richmond and Allegheny, and the Louisville, New Orleans and Texas Railroads; 1886 to 1887, Engineer in Charge of the Lynchburg and Durham Railroad; and 1887 to 1888, Principal Assistant Engineer on the Roanoke and Southern Railroad. In 1888, he retired from active work in his profession and returned to his home at Houston, Va., where he lived quietly until his death on May 9th, 1915.

Colonel Derrick's professional career during his thirty-seven years of railroad service was typical of the life of the civil engineer in the United States during that period when railroad building was most active. The greater part of the Profession was engaged on preliminary survey, location, and construction, and as soon as one railroad was nearing completion, they went to another. It was a hard but interesting life, useful to the country and the Nation, but it seldom led to fame or fortune for the individual. Colonel Derrick's retirement from active practice coincided with the discontinuance of active railroad construction in the United States.

In writing of Colonel Derrick's personality, those who knew him best speak of him with affection and respect, dwelling especially on his modesty and simplicity, his courage, his high-mindedness, and on the essential dignity of his character. He was intensely loyal to the South, where he preferred to remain in quiet seclusion rather than seek a wider environment where his personality and endowments would have gained for him the high professional position which he deserved. In social circles his keen wit and humor were always evident and delightful, and as a host, he was the soul of geniality, courtesy, and hospitality.

Colonel Derrick is survived by his widow, who was Miss Martha Cosby, and two daughters and three sons. Two of the sons, Guy H. and Clarence Derrick, and a grandson, John Russell Derrick, are members of this Society.

He was a Member of the National Geographical Society, and for many years a faithful officer and communicant of St. John's Protestant Episcopal Church, at Houston, Va.

Colonel Derrick was elected a Member of the American Society of Civil Engineers, on October 5th, 1887.

JOHN WILLIAM EBER, M. Am. Soc. C. E.*

DIED AUGUST 18TH, 1915.

John William Eber was born at Jersey Shore, Pa., on July 14th, 1871. He received his early education at, and was graduated from, the High School at Williamsport, Pa. During the next five years, he studied higher mathematics and engineering with a tutor and independently.

Mr. Eber entered the employ of the West Shore Railroad (New York Central and Hudson River Railroad, Lessee), on February 28th, 1890, serving as Rodman, Transitman, Draftsman, and General Assistant to the Assistant Engineer of the Buffalo Division, at Syracuse, N. Y., until November, 1896, when he was appointed Assistant Roadmaster. He was engaged in that capacity and, later, as Supervisor of Track until 1902, when he assumed a similar position with the New York Central and Hudson River Railroad at Albany, N. Y., having charge of the reconstruction and enlargement of the West Albany yards and many works of a similar nature. In April, 1904, Mr. Eber was appointed Engineer of the Rome, Watertown and Ogdensburg Division, and in the following year Assistant Engineer of Maintenance of Way and Engineer of Track, with jurisdiction over 3 785 miles of road. From 1909 to 1910, he served as Assistant Superintendent of the New York Central Stockyards, at Buffalo, in charge of reconstruction, design, maintenance, and on special work.

In 1910, Mr. Eber was appointed Superintendent of the Adirondack Division of the New York Central between Utica, N. Y., and Montreal, Ont., Canada, in charge of both maintenance and operation. In May, 1912, he left the service of that company to accept the position of General Superintendent of the Toronto, Hamilton and Buffalo Railway, at Hamilton, Ont., Canada. On January 1st, 1913, he was promoted to be General Manager of the latter road.

Owing to ill health, Mr. Eber resigned his position in June, 1915. He died on August 18th, 1915, and was buried at Williamsport, Pa.

He possessed qualities for thoroughness, had fine tact, and, throughout his career, was a favorite with all those with whom he came in contact.

Mr. Eber was elected a Member of the American Society of Civil Engineers on September 4th, 1907.

* Memoir prepared by F. F. Backus, Esq., Assistant to the President, Toronto, Hamilton and Buffalo Railway Company, Hamilton, Ont., Canada.

EMORY ALEXANDER ELLSWORTH, M. Am. Soc. C. E.*

DIED DECEMBER 8TH, 1915.

Emory Alexander Ellsworth, the son of John F. and Maria (Lawrence) Ellsworth, was born in Hardwick, Mass., on August 3d, 1852.

He was educated in the schools of his native town, and was graduated from the Massachusetts Agricultural College at Amherst, in the Class of 1871, this being the first class to be graduated from that institution.

In 1872, Mr. Ellsworth began work as an Assistant on the construction of the water-works for Holyoke, Mass., and from 1873 to 1876, was a member of the firm of Davis and Ellsworth, engaged in general engineering practice in Holyoke.

In 1876, he left engineering and became a traveling salesman, following this work until 1880, when he returned to the Profession as Section Engineer for the New Haven and Northampton Company, now the New York, New Haven and Hartford Railroad Company, on construction of the then new railroad from Northampton to Turners Falls, Mass.

From 1881 to 1884 he was Principal Assistant Engineer for D. H. and A. B. Tower, Architects and Mill Engineers, at Holyoke, Mass.

In 1884, Mr. Ellsworth was elected City Engineer of Holyoke, Mass., which position he held for five years. As City Engineer, he was also Engineer for the Water-Works. While he occupied this position he designed several city school and fire department buildings, also three reservoir dams for the water supply system.

In 1890, he resumed private practice in Holyoke which he continued until 1901, when Mr. John J. Kirkpatrick became associated with him under the firm name of Ellsworth and Kirkpatrick. This arrangement continued until May 1st, 1907, when Mr. Kirkpatrick retired.

After Mr. Kirkpatrick's withdrawal, the firm became Ellsworth and Howes, with Mr. Lyman R. Howes as Junior Member. This partnership continued until Mr. Ellsworth's death, which occurred at his home in Holyoke, Mass., on December 8th, 1915.

He was the Architect for several of the buildings of the State Insane Hospital at Northampton, Mass., also for the Massachusetts College at Amherst, Mass., and designed several paper and textile mills.

Mr. Ellsworth was of a quiet, unassuming disposition, and a hard worker. He was devoted to his family and home.

He was twice married, first to Miss Lucy J. Bradford, who died in 1900, and, in 1902, to Miss Carrie Meach, who survives him. By his

* Memoir prepared by J. W. Tower, M. Am. Soc. C. E.

first marriage there were two sons and one daughter, all of whom are living, together with five grandchildren.

He was a Member of the Boston Society of Civil Engineers and a Charter Member of the Bay State Club of Holyoke.

Mr. Ellsworth was elected a Member of the American Society of Civil Engineers on June 1st, 1904.

ASA BETTS FITCH, M. Am. Soc. C. E.*

DIED MARCH 11TH, 1916.

Asa Betts Fitch, the eldest son of Charles and Elizabeth Marilla (Betts) Fitch, was born on a farm near Ellsworth, Ohio, on June 17th, 1840. He was a lineal descendant of Thomas Fitch, the Colonial Governor of Connecticut under King George III.

In 1847, his parents removed to Woodstock, Ill., where he studied at public and private schools until 1857, when the family removed to Stacyville, Iowa. In this new country advanced educational facilities were lacking, and during 1858 and 1859, Mr. Fitch worked on the farm during the summer and taught country schools in the winter. In 1860, he taught school at Mitchell, Iowa, and prepared for college with a private tutor.

The call to arms in the Civil War, however, over-ruled, in his mind, all private affairs, and he volunteered. He took part in raising a company at Stacyville, Iowa, in which he enlisted as private, becoming First Sergeant on its organization. This company became H in the Fourth Iowa Cavalry, in which regiment he was in active service in the field until the end of the war, serving in many engagements in Missouri, Kansas, Arkansas, Mississippi, and Alabama, and never off duty. He was wounded in action in command of his company at Marianna, Ark., was rapidly promoted until he became Captain, and was recommended for promotion to Brevet-Major by his Brigade, Division, and Corps Commanders "for great gallantry" in battle. During the last year of the war, Captain Fitch was detached as Aide and then as Acting Assistant Quartermaster-General on the staff of General Winslow, commanding brigade and division.

Among the many campaigns in which he was engaged were those in which Vicksburg was taken, Price's army was destroyed in Missouri and Kansas, Forrest was defeated in Mississippi, and Wilson's column of cavalry swept through Alabama and Georgia, the greatest and most successful cavalry campaign in history.

Captain Fitch was an ideal soldier—tall, straight, of fine military bearing; decided in speech, yet of few words; careful of his men yet keeping his hand sternly on the reins of duty; bold yet not reckless; always ready for orders and instantly zealous in their execution. If he had chosen a military career, he must have become an admirable soldier and a distinguished commander.

At the end of his military service, Captain Fitch decided that he was too old to enter college, but this did not alter his determination

* Memoir prepared by the Secretary from information furnished by the family and on file at the Society House.

to become a Civil Engineer, although this decision meant an uphill course of study and practical work at the same time, and resulted in his being a student all his life.

Having suffered much from malaria during the last year of his Army service and being unfit for work, he spent the summer and winter of 1865-66 in visiting relatives in New Hampshire and hunting in Northern Iowa.

In the spring of 1866, Captain Fitch entered the employ of the National Express Company in Cincinnati, Ohio, as "Route Agent", under his old Commander, General Winslow. With the idea of carrying out his plan of becoming a Civil Engineer, however, he applied to the late Grenville M. Dodge, Hon. M. Am. Soc. C. E., Chief Engineer of the Union Pacific Railroad, then being commenced, for a position with the Engineering Corps. Gen. Dodge appointed him Division Engineer of the road in Nebraska, but, after consideration, Captain Fitch declined the appointment to go with Gen. Winslow on the Vandalia Railroad then being projected. Work on this road was begun in the Spring of 1867, and Captain Fitch continued in the employ of the Company under Col. Carswell McClellan and the late E. C. Rice, M. Am. Soc. C. E., until the road was purchased by the Pennsylvania Railroad Company in 1869, having been promoted from Chainman to Assistant Engineer.

Captain Fitch then entered the service of the Evansville, Terre Haute and Chicago Railroad Company as Transitman on preliminary survey and location, and, in 1870, when construction was begun, he was put in charge of a division of 22 miles, which included a bridge across the Wabash River, at Clinton, Ind. When the road was completed and opened in 1871, he was made Chief Engineer, Roadmaster, and Paymaster. He retained these positions, planning the yards, shops, etc., at Danville, Ill., and Terre Haute, Ind., laying out short branch lines to coalfields, etc., until early in 1873 when he leased 80 acres of coal lands east of the Wabash River, near Clinton, Ind. For a year the enterprise proved successful, but the financial panic and strikes of that period made the situation desperate for the owners, and to control competition, Captain Fitch organized a combination of all the Clinton mines, of which he became President and Sales Agent. After five years of exacting labor, he accepted an offer for his entire coal interests which left him without any reward for his work. During this strenuous period he had also laid out and supervised the construction of 4 miles of gravel road running west from Clinton, which was a marked success and attracted much attention among road builders throughout the State of Indiana.

In 1878, Captain Fitch served one term as City Engineer of Terre Haute, Ind., and in that capacity built sewers and gravel streets,

re-surveyed the City, and marked the streets with permanent monuments.

In 1879, after building a bridge over White River on the Evansville and Crawfordsville Railroad, he was appointed Chief Engineer of the Terre Haute and Southeastern Railroad, which position he retained for nearly two years. He located and built an extension of 14 miles from Clay City to Worthington, Ind., extended the road to connections with other roads in Terre Haute, and built several short branches to coal mines in the vicinity.

Late in the summer of 1880, Captain Fitch was made Chief Engineer on the extension of the Terre Haute and Logansport Railroad from Logansport to South Bend, a distance of 67 miles, and it was on this work that his reputation as a Location Engineer of much more than average ability, was made.

Before the completion of his work on the Logansport Extension, he was appointed Superintendent of Construction of the Government Building at Terre Haute, Ind., but a political change at Washington compelled him to resign before he had completed the work.

During 1885 and until the autumn of 1886, Captain Fitch was engaged in a survey for 8 miles of railroad through the coalfields of Illinois; on a report on a railroad site and coalfield in the Black Hills of South Dakota; and on an investigation of gold and silver mines in Mexico, for various financial interests.

During the college term of 1886-87, he occupied the Chair of Civil Engineering at Rose Polytechnic Institute, taking the place of a professor who had resigned just before the term began. Although without college training himself, he was very successful in this work and was urged to continue it, but he declined, and late in 1887, he was engaged to re-measure some heavy rock excavation on the Nashville and Knoxville Railroad in Tennessee. In 1888, he was made Chief Engineer of that road, building the bridge across the Caney Fork of the Cumberland River and completing the road to Cookville.

During the winter of 1888-89, Captain Fitch made an examination and estimate for the completion of 80 miles of railroad from New Castle, Pa., to Akron, Ohio, for some New York financiers. Early in 1889, he returned to the Mission Coal Field in Indiana, in which he had never quite lost his early interest, and obtaining options for the purchase of the lands covering those fields, he made plans and estimates for a railroad to tap that region. He submitted all the data to the General Manager of the Illinois Central Railroad and while the officials of that road were considering the proposition, Captain Fitch went to Texas and made an examination and report on an extension of the Austin and Northwestern Railroad from Burnet to Granite Mountain, which was afterward constructed. On his

return to Indiana, he found that the Illinois Central had decided not to use his plans, and, his options on the Mission Coal Field having about expired, he sold them to the Consolidated Coal Company of St. Louis, Mo., and that Company afterward developed the field on his plans.

In 1889, after making a preliminary survey for a railroad up the valley of Green River, from Henderson to Bowling Green, Ky., about 100 miles (which was never built), Captain Fitch went to Nevada, in the interests of some New York investors, to report on a coal property. This work was followed by reconnaissances for the relocation of the Terre Haute and Indianapolis Railroad in the vicinity of Rockville, Ind., and also for the Vandalia Line, both of which were afterward built.

Early in 1890, Captain Fitch went to Chattanooga, Tenn., to examine and report on coal land in Southern Tennessee, and remained to take charge of various financial projects for Mr. Jere Baxter, of Nashville, Tenn. Among these projects were the management of Lookout Inn, a railroad running from the city to the Inn, a coalfield on Lookout Mountain, and a Belt Railroad around Chattanooga. He was very successful in the management of these interests, but preferred engineering, and after Mr. Baxter's return, Captain Fitch went to Florida to study the occurrence and working of phosphate deposits in that State.

In the fall of 1890, he went to Salt Lake City, Utah, to examine and report on the Utah Central Railroad for English financiers. In 1891, Captain Fitch examined and located an extension of the Danville, Olney and Ohio Railroad from Sidell to Chicago, with an estimate of the cost, for some promoters, but the scheme failed.

In the latter part of 1891, he was asked to report on some vitrified brick paving contracts which had been made by the Common Council of Terre Haute, by a Committee which was investigating the matter, and after much persuasion, Captain Fitch was prevailed on to accept the appointment of City Engineer and finish the work. At this time vitrified brick paving was a novelty, and as there were no standards to work by, he had to "blaze the way". His work was successful, but as soon as it was completed he resigned.

In 1893, Captain Fitch went to Magdalena, N. Mex., to erect hoisting machinery in the Graphic Mine at that place, for Terre Haute and Philadelphia promoters. He expected to remain only a few weeks, but stayed eleven years, removing his family there in December, 1896. After developing the property, he, in June, 1896, leased the entire plant, built a smelter for his ore, and a tram-road to the mine. In the summer of 1902 the ore in the property was apparently exhausted, and the smelter was shut down. After many financial and legal vicissi-

tudes, in the summer of 1903, replacement ore was discovered. This increased greatly the value of the Graphic property, and, on March 30th, 1904, it was sold to the Sherwin-Williams Paint Company, of Cleveland, Ohio, and Captain Fitch removed his family to California.

In 1905, they settled in Hollywood, where Captain Fitch organized the Temescal Rock Company of which he was President and Manager until his death on March 11th, 1916.

He was married on January 15th, 1868, to Miss Amelia Pauline Grimes, the daughter of James and Mary Ann (Tobin) Grimes, old and prominent residents of Portsmouth, Ohio, and she, with four sons and three daughters, survives him.

Captain Fitch was peculiarly a home body, loving his home and family, and only his intimate friends knew the broad scope of his information, his splendid mental poise, his rare executive ability, his kindly management of men, and his genial nature.

He was a member of the Masonic Fraternity, the Loyal Legion, and of St. Stephen's Protestant Episcopal Church, of Hollywood, Cal.

Captain Fitch was elected a Member of the American Society of Civil Engineers on March 6th, 1884.

WILLIAM WALLACE FOLLETT, M. Am. Soc. C. E.*

DIED DECEMBER 28TH, 1915.

William Wallace Follett was born in New Sharon, Me., on September 22d, 1856, and was graduated from the University of Michigan in June, 1881. His college course was completed on a minimum of borrowed money, and only a strong will power, which characterized all his life of hard work, enabled him to persist in order to obtain his degree.

After his graduation, Mr. Follett began work on the location of the New Orleans and North Eastern Railroad, in the swamps of Louisiana, where he had to lead a crude existence, combating unpleasant forces in an unwholesome climate. For 3 years he was in charge of a "Residency" of 13 miles of grading on that road. He next took charge of the construction of a highway and railroad bridge across the Red River for the Vicksburg, Shreveport and Pacific Railroad. Afterward, for the same Corporation, he had charge of the erection of some roundhouses, depots, etc., and also of field work in remodeling the Shreveport Yards. Later, he was engaged in levee work and the study of rectifying the channel of the Red River below Shreveport.

In March, 1886, Mr. Follett was employed by the Atchison, Topeka and Santa Fé Railroad on topography, and afterward was placed in charge of the construction on one of the divisions of that line. He completed the grading on two divisions of 15 miles each, and established the grade line on the Ottawa-Emporia Division. He also located all except about 16 miles of the Denver-Pueblo Line, and made surveys for the Denver Terminals and for about 350 miles of preliminary lines and about 60 miles of location in Colorado.

In November, 1887, Mr. Follett was given command of the construction forces, finishing up the yards, etc., of the Denver-Pueblo Line, and in May, 1888, he took charge of the drafting-room of the Fuel Department, at Topeka, Kans. In 1889, he returned to Denver and made topographical surveys, plans, and an estimate of cost for a summer resort, the work for which was carried out and proved successful. In that and the following year, he was with the United States Geological Survey as Division Engineer on irrigation surveys. He made the preliminary surveys, maps, plans, and estimates of cost for the "International Dam" and reservoir at El Paso, Tex., completing also the reconnoissance of the Rio Grande drainage, hunting and segregating reservoir sites.

In 1890, Mr. Follett took up general engineering work at Denver and located 25 miles of a rather difficult line for the Perry Park Rail-

*Memoir prepared by Stuart Henry, Esq.

road. From 1890 to 1892, he was Assistant Engineer of the Artesian Underflow Investigation, United States Department of Agriculture. He next devoted his energies to general engineering work, examinations, and reports of irrigation schemes, a design for a private water supply system, and also completed extensive mineral surveys in New Mexico.

In 1893 he located a canal for Col. Nettleton on the Yaqui River. In 1894, he was appointed Engineer in charge of the "Delgany Street Public Sewer Extension," in Denver, and during the next year located a railroad from Flagstaff, Ariz., up the Grand Canyon. In 1896 he was Consulting Engineer for the Dodge City (Kansas) Irrigating Canal, and wrote an exhaustive report on the water conditions of the Upper Arkansas River in Kansas.

Mr. Follett was appointed, in March, 1897, Consulting Engineer for the International (Water) Boundary Commission, United States and Mexico, and held that position until April, 1900, when he resigned and went back to Denver, whence he was immediately called to Topeka to make an appraisal of the water-works. He next put in a pipe line, 20 miles long, to furnish water to Prescott, Ariz., and also did work for Gen. Palmer at Colorado Springs.

He was re-appointed Consulting Engineer for the International Boundary Commission on the Mexican border early in 1902, which position he held until July, 1914. In 1906, he was selected as Consulting Engineer for the United States Reclamation Service. In 1907, he had expert charge of an irrigation project on the Pecos River for the United States and Mexico Trust Company. He made a hydrographic investigation, in 1913, for the El Paso and Southwestern Railroad, for the supply of water for its pipe lines near Carrizozo, N. Mex.

In June, 1914, the University of Michigan conferred on Mr. Follett the honorary degree of Master of Engineering for eminence as an irrigation engineer and especially for his work on the Mexican Boundary, for no one could speak with such authority as he on the subject of the equitable distribution of the waters of the Rio Grande. At about this time he was appointed Consulting Engineer for the Elephant Butte Water Association and the El Paso Water Users Association, which together had plans for irrigating some 185 000 acres of land under the Engle Dam, then nearing completion, but he was unable to accept this position on account of ill health.

Mr. Follett delivered many public addresses in the West on irrigation, water supplies, and the rights of States, different valleys, and communities to the use of various western waters. Throughout the Rocky Mountain region he was a recognized authority on this large and difficult subject. He was frequently called to Washington for expert advice.

When Mr. Follett stepped down from the International Boundary Commission on July 1st, 1914, his days had long been numbered. Many medical experts in all parts of the country were consulted as to his distressing physical condition, but with little avail. He weakened rapidly, and died at his home in El Paso, Tex., on December 28th, 1915. In July, 1915, a letter to the writer contained the following:

"I am all in, old Pard, and you will never see me again. How long I will linger I can't say. I suffer all the time, and won't be at all sorry when the time comes for me to go."

Although his years were frequently beset with obstacles which would have daunted feebler souls, Mr. Follett never faltered in his dogged pursuit of whatever task or aim was before him. Blessed with a tough physique as a young man, he readily sacrificed himself to any conditions of toil, often living under unsanitary conditions and with improper food, as he whipped himself on to his many separate tasks.

These early hardships and deprivations after a time affected his health, so that the last twenty years of his busy career were passed under an almost steady strain of pain, more or less acute, while he hung on to his duties with unflinching courage. The writer has seen him at work over reports day after day with a big bottle of ugly looking medicine standing by his manuscript. A frequent dose from the bottle would assuage his suffering sufficiently to enable him to prosecute his labors.

This physical and moral heroism was equalled by his incorruptible honesty as to fact and opinion. He could not be bought or influenced. He declined many a promising job, because he did not believe it served the truth or an upright end.

Mr. Follett was a keen judge of human nature, and possessed a lively sense of humor. He would resolutely pass off most despairing situations in his work with a brave jest, and lose little delay in regrets and repining. He was fond of chess, and derived the utmost pleasure from attending the theater whenever he was in New York City.

In 1888, he was married to Helen Jordan at Pueblo, Colo., who, with two sons, William L., Hydrographer, United States Reclamation Service, at El Paso, Tex., and Leslie C., aged sixteen, survives him.

Mr. Follett contributed in high measure to the renown of the Engineering Profession. He had resistless energy, was relentless in logic, and never shirked the smallest detail. He was a valuable, courageous, and public spirited man, wholly devoted to his family, and would divide his last crust with any friend.

Mr. Follett was elected a Member of the American Society of Civil Engineers on July 5th, 1893.

CHARLES FRANCIS, M. Am. Soc. C. E.*

DIED APRIL 29TH, 1914.

Charles Francis was born in Lowell, Mass., on August 10th, 1842. He was the son of the late James Bicheno Francis, Hon. M. Am. Soc. C. E., the eleventh President of this Society.

Charles Francis was reared in Lowell, and pursued his studies there until 1860, when he entered Harvard University, from which he was graduated with the Class of 1864. In 1861, while at Harvard, Mr. Francis enlisted as a soldier of the Union Army and served his country as a member of Company F of the 44th Massachusetts Regiment, receiving his honorable discharge after one year's service.

After Mr. Francis was graduated from Harvard, he determined to follow his father's profession, and as he wished to become familiar with machine-shop practice, he entered the Lowell Machine Shop. After serving his apprenticeship as a machinist, he spent several years in his father's office, and was associated with him on some of his important work.

Later, Mr. Francis went to Chicago where he remained until the great fire of 1871, at which time he removed to California. There his training as a hydraulic engineer was of great value to him, and for nine years he was associated with various mining enterprises in that State.

From 1879 until 1883, Mr. Francis was employed by the Mexican Central Railroad, with headquarters at Guadalajara, Mexico, working under the direction of the late A. M. Wellington, M. Am. Soc. C. E.

In 1883, Mr. Francis returned to Lowell where he was again associated in business with his father until 1889. In that year he accepted employment with the Federal Government as Engineer in charge of the construction of a dam at Rock Island Arsenal.

It was at this time that Mr. Francis' acquaintance with Davenport, Iowa, began, as he resided there while in charge of the Government work, and, at its completion, his attachment to that city was such that he decided to make it his permanent home. He immediately became prominent in the public affairs of the city, serving as Commissioner of Public Works from 1891 to 1892, and afterward practising as a Construction Engineer. He also served as a member of the Iowa State Board of Health for seven years.

Soon after he came to Davenport, Mr. Francis became interested in the possibilities of the hydro-electric development of the Mississippi River at that point, and was associated with the Davenport Water Power Company as Chief Engineer for twelve years. During this

* Memoir prepared by William H. Kimball, M. Am. Soc. C. E.

time he made a careful study of the project, and his investigation of it showed the painstaking care that was characteristic of his work.

Mr. Francis was prominent in various organizations in Davenport, served on many local boards, and always gave his services generously. Everything he undertook was well done.

He was a member of the Masonic Fraternity, a Knight Templar, and a Noble of the Mystic Shrine.

As a member of the Protestant Episcopal Church, Mr. Francis served on the Vestry and, for some time before his death, as Junior Warden. He was also President of the Men's Club of the church, and gave generously of his time and means for its support.

Mr. Francis' travels, his broad general knowledge, his alert mind, and his interest in people made him a most companionable man. He was one of those men for whom the Profession of Engineering meant an abiding, life-long interest, not because of its pecuniary reward, but because he was proud to belong to it. He saw the various ways in which his engineering knowledge could serve the community, and he gave freely of this knowledge.

When the American Society of Civil Engineers visited the Panama Canal, Mr. Francis was a most enthusiastic member of the party, and on his return gave several illustrated talks describing the trip and the work.

In 1869, Mr. Francis was united in marriage to Miss S. C. Crosby, of Lowell, Mass., who, with an only daughter, Fanny C., survives him.

Mr. Francis was elected a Member of the American Society of Civil Engineers on May 4th, 1892.

ALEXIS HENRY FRENCH, M. Am. Soc. C. E.*

DIED MAY 3D, 1915.

Alexis Henry French was born at North Weymouth, Mass., on May 2d, 1851, and was the son of Henry J. and Lucy H. (White) French.

He received his education in the public schools of his native town, and having been graduated from the Weymouth High School, became an apprentice in the office of Shedd and Sawyer, at that time a well-known engineering firm of Boston, Mass. In 1871, he was appointed an Assistant to Mr. George Tyler, the Engineer and Superintendent of Streets of Brookline, Mass.

Later, Mr. French entered the Massachusetts Institute of Technology as a special student, being associated with the class of 1873. He spent two detached years at the Institute, all the time that he felt he could afford.

On the retirement of Mr. Tyler in 1875, Mr. French was given the use of an office in the Town Hall of Brookline under an arrangement by which he did engineering work for the town and for private parties. This arrangement continued until 1894, when the salaried office of Town Engineer was created, and he was appointed to and held the office until his death.

The title of Town Engineer indicates but slightly the extent and importance of Mr. French's work. As a town, Brookline is unique in having probably the largest population of any municipality retaining the old New England form of town government; and it is also very wealthy, so that its public works are similar to those of a city several times its size. Such a condition requires engineering services of a high order, and this condition Mr. French very ably met.

He saw Brookline grow from a population of 8 000 to one of 30 000. Nearly the entire sewerage and drainage systems of the town, involving a large expenditure and the overcoming of many technical difficulties, were constructed under his direction. The engineering work required in laying out and constructing the Park and Playground System was extensive, and the result is a visible and constantly appreciated monument to the skill with which Mr. French carried out his own designs and those of others. The stone arches across the Park waterway were nearly all designed in his office, and were all built under his direction. Some of these have attracted the attention of

* Memoir prepared by Edward W. Howe, M. Am. Soc. C. E., from information furnished by Henry F. Bryant, Esq., Brookline, Mass.

many engineers, architects, and artists, on account of their great beauty and artistic setting.

The laying out of new streets, the designing and construction of bridges, and the abolition of grade crossings also constituted a large part of the work requiring the Town Engineer's attention. Besides, there were the innumerable minor matters which always make up a large part of the work of the municipal engineer. Mr. French's large experience and wide observation enabled him to meet all these requirements in a way that won him the confidence and respect of all those in any way connected with town affairs, as well as of his contemporaries in the Engineering Profession.

Mr. French had suffered for several months from arterio-sclerosis and had been obliged, in the latter part of 1914, to give up his usual business, except such as he could attend to at his home. His death, which occurred on May 3d, 1915, came quite suddenly, he having celebrated with his family only the day before his sixty-fourth birthday; the occasion was a happy one, and he seemed to be in excellent spirits. He is survived by his wife who was Miss Alice Blanchard Loud, of Weymouth, Mass., whom he married on January 14th, 1880.

Mr. French was a member of the following organizations: The Boston Society of Civil Engineers, of which he was President in 1900; the Massachusetts Highway Association, the Engineers' Club of Boston, the Appalachian Mountain Club, of which he was a Past-President, and several local societies, in all of which he took an active interest. He was an active member of the Harvard Congregational Church, in Brookline, having been for a long time a member of its Prudential Committee.

Mr. French was a great lover of outdoor life, and many of his vacations were spent in the forests and on the mountain tops of New England. Many years ago he took up photography as a recreation, and practised the art with great skill and success. His collection of mountain views presented to the Appalachian Mountain Club have been highly praised for their execution and artistic merit.

Few men have left behind them a more enviable record of integrity and devotion to duty and a willingness to give of themselves for the advancement of mankind. As a public official, his quiet, unassuming methods, unfailing courtesy, and freedom from all political or personal influences tending to divert him from what he felt was his duty to those whom he served, furnish a good example which those called on to serve their fellow citizens in a like capacity may well follow.

Mr. French was elected a Member of the American Society of Civil Engineers on June 6th, 1894.

EDWARD GRAY, M. Am. Soc. C. E.*

DIED OCTOBER 2D, 1915.

Edward Gray, the son of Mr. and Mrs. James Gray, was born at Belfast, Ireland, on December 6th, 1875. When he was two years old, his parents came to the United States and settled in Princeton, Ind., where the boy was reared and educated in the public schools.

He began his professional career in 1896, when he entered the office of E. E. Watts, County Surveyor and City Engineer of Princeton. In 1897, Mr. Gray was appointed Deputy County Surveyor, and had charge, during 1897 and 1898, of dredging operations for irrigation, surveys for roads, and other municipal work.

In 1898, he was appointed Draftsman on the Louisville, Evansville, and St. Louis Consolidated Railway, and, in 1899, was made Assistant Engineer in charge of location and maintenance.

In 1901, Mr. Gray entered the employ of the Southern Railway Company as Assistant Engineer. He remained with the Company for 11 years, serving as Principal Assistant Engineer, Engineer of Maintenance of Way, and Chief Engineer of the St. Louis and Louisville Division, with headquarters at St. Louis, Mo.

In 1912, his health began to fail, and he went to Colorado where he became Chief Engineer of the Florence and Cripple Creek Railroad. He was also Chief Engineer of the Automobile Highway up Pike's Peak, and held both these positions at the time of his death which occurred at his home in Colorado Springs, Colo., on October 2d, 1915. He is survived by his widow, Mrs. Lillian Branham Gray, and one daughter, Janice.

Mr. Gray was a young man of great promise. Although only 37 years old at the time of his death, he had attained high rank in his profession and had held, for a number of years, positions of such responsibility as seldom fall to so young a man.

He was possessed of a very pleasing personality which won for him friends wherever he went or in whatever task he was engaged, and he held at all times the respect and confidence of those for whom he worked, as well as of those who worked for or under him. Such a combination of capacity, industry, affability, and absolute integrity is rarely found in one man, and Mr. Gray's death was not only a severe blow to his family and friends, but a great loss to the Engineering Profession to which he was devotedly attached.

Mr. Gray was elected an Associate Member of the American Society of Civil Engineers on May 1st, 1907, and a Member on December 6th, 1910.

* Memoir prepared by the Secretary from information on file at the House of the Society.

BERNARD RICHARDSON GREEN, M. Am. Soc. C. E.*

DIED OCTOBER 22D, 1914.

In the death of Bernard Richardson Green, a former Vice-President of this Society, this country lost an able civil engineer and a conscientious, devoted, and modest public servant; his associates, a wise and experienced counsellor; the community in which he lived, a generous, unselfish, and public-spirited citizen; and his friends, a loyal, frank, and gentle companion.

He died at his home in Washington, D. C., on October 22d, 1914, in the seventy-first year of his age, having been born on December 28th, 1843. He is survived by his widow, Julia Lincoln, to whom he was married in 1868, and by one daughter and three sons.

His birthplace was Malden, Mass., the home of his parents, Ezra and Elmina (Richardson) Green. From the public schools, where he obtained his earlier education, young Green went to the Lawrence Scientific School, which he attended as a student in Engineering from 1861 to 1863.

In 1863 he entered the service of the United States Government, in which he remained for more than half a century. He began his service as a Civilian Assistant, under the Corps of Engineers of the Army, and for 14 years he was mainly engaged in the construction of permanent sea coast defenses on the coasts of Maine, New Hampshire, and Massachusetts. During this time he acquired a thorough knowledge of governmental methods of procedure in public works, a knowledge which is of great importance in government work, and the lack of which has often hampered civil engineers who have not learned to unravel the mazes of red tape so as to keep it from stifling the economy and efficiency which is the pride of our Profession.

Although at first serving under the Corps of Engineers of the U. S. Army, and associated with it for the greater portion of his career, Mr. Green was never a member of that Corps, but always a civilian. Therefore, the military title by which he was sometimes addressed was erroneous.

In 1877 his activities were transferred to Washington, where he distinguished himself as an architectural engineer in the building of many notable government structures. The earlier of these were erected under the general direction of Lieut.-Col. Thomas Lincoln Casey, Corps of Engineers, U. S. A., who later became Chief of Engineers. From then until Gen. Casey's death Mr. Green was his Assistant.

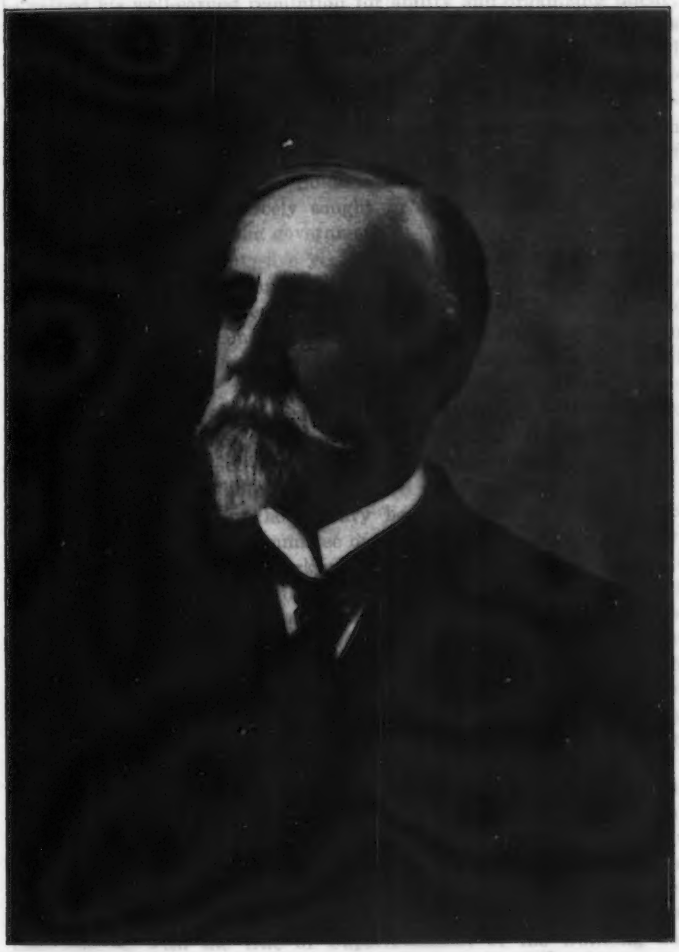
Mr. Green began his duties in Washington at a period when official architecture did not concern itself with the homogeneity of design

* Memoir prepared by the following Committee: O. H. Tittmann, George F. Swain, and Clifford Richardson, Members, Am. Soc. C. E.

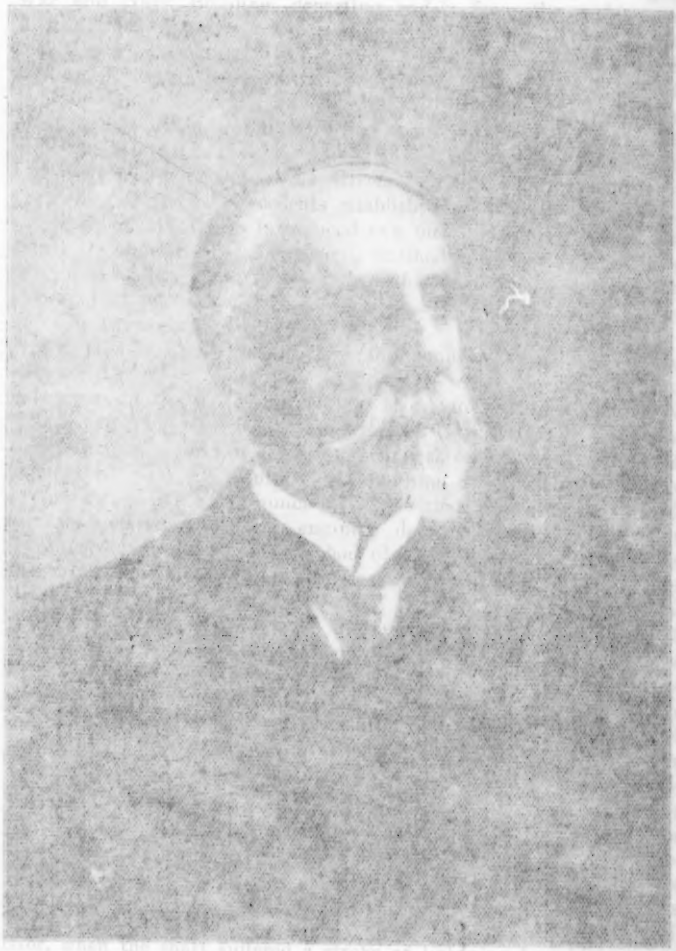
for government buildings there. One of these, known as the State, War and Navy Building, departing widely from the architectural spirit of the Treasury Building and the White House, but flanking the latter, had been commenced, and to Mr. Green was assigned the task of completing the huge structure. The design had been already sanctioned and adopted by the government, and in part carried out by the construction of the south and a part of the east wings of the building, before Col. Casey was charged with the prosecution of the work which he entrusted to Mr. Green. Although hampered by existing contracts and by precedents established in the completed part of the building, Mr. Green introduced new methods and, by his circumspection and efficient management, finished the structure at a cost so much smaller, proportionally, than that incurred in the construction of that portion of the building already finished, as to save the government several millions of dollars.

Mr. Green's name is also inseparably connected with the construction of the Washington Monument. The weather marking of the lower part of the monument plainly shows to this day how far the obelisk had progressed before the National Government undertook its completion. The site of the monument was on low, alluvial ground, and before work was resumed, it was found necessary to strengthen materially the old stone foundation under that part of the shaft built by the predecessors of the engineer destined to complete it. In searching for a satisfactory method of accomplishing this, engineers throughout the country, both in the U. S. Service and in civil life, were invited to make suggestions. Of those submitted, however, the method proposed by Mr. Green was adopted as the most practical. He undermined the old stone foundation, excavating narrow sections, on opposite sides at a time, until there had been provided a broad and deep concrete base, amply sufficient for the entire load of the shaft when completed. Although the portion already built was found to be nearly 2 in. out of plumb, the operations under the footings were balanced so nicely that on their completion, the error was substantially corrected and the shaft was almost perfectly vertical. There has been no settlement since. Although Mr. Green was not directly in charge of these operations, he nevertheless maintained a general supervision of the entire completion of the monument. He designed the very unique and extremely light marble pyramidion which completes the obelisk, also many less important details of its construction; and, later, when the shaft suffered a stroke of lightning by which one of the pyramidion stones was dislodged, he supervised the repairs and the installation of an entirely new system of protection from lightning, which has proved entirely satisfactory.

Mr. Green's connection with the completion of the State, War and Navy Building is mentioned in order to illustrate the manner in which



Bernard R. Green.



he gained his well-earned reputation for ability and organizing talent; his connection with the building of the Washington Monument is mentioned because his relation to the completion of that beautiful shaft should be remembered by the thousands of his countrymen who visit it. During the time that he was acting as assistant and professional adviser to Col. Casey, however, he supervised the construction of other buildings, such as the Army Medical Museum and some of the principal buildings in the Soldiers' Home. At the same time, his professional advice was freely sought in Washington and elsewhere in matters unrelated to the government.

During the last twenty-five years of his life his energies were devoted mainly to the building for the Library of Congress, although his professional service was extended to many other fields as well. He was in charge of the construction of the Library Building, and later superintended all the maintenance and repairs, and the expansion of its book-stack capacity. Many of the interior appointments, and especially the scheme of storing and delivering books, give testimony to his originality and thoroughness. The whole structure, as a library plant, has been a model for most of the library construction undertaken since.

His connection with the Library began in the spring of 1888, when he was asked by a committee of Congress to examine and report on the condition of the work already accomplished, and estimate the probable cost of the building if carried out in accordance with the plans then in force. After his report had been received and considered he was invited to take charge of the construction of the building from modified plans to be prepared by an associated architect along the lines suggested in his report. In the autumn of 1888, however, Gen. Casey having returned to Washington as Chief of Engineers, the entire control of this undertaking was placed in his hands by Act of Congress.

Gen. Casey immediately entrusted the execution of the work to Mr. Green, and he was in direct responsible charge of it until Gen. Casey's death early in 1896. Thereupon, by Act of Congress, Mr. Green was appointed to the entire control of the construction. The building was finished in 1897, well within the estimated time and cost, and with remarkable economy. He was then given charge of its maintenance, with the title of Superintendent of the Building and Disbursing Officer of the Library, and this office he held at the time of his death.

Among his other activities in later years were his supervision of the construction of the Pennsylvania State Capitol at Harrisburg, Pa., and an examination and report on the new State Capitol at Jackson, Miss.

His portrait makes a description of his personal appearance unnecessary. He was a man of commanding presence, 6 ft. tall, and well-proportioned. He was as gentle in his manner as he was firm of purpose. He loved music, and gave freely of his time and means to promote a taste for its refining and ennobling influence. He was long associated with the Washington Choral Society, to which he gave his help and counsel.

He was a Trustee of the Corcoran Art Gallery, was active in the scientific societies of Washington, and was one of the founders of the flourishing Washington Society of Engineers. In 1899 he was President of the famous Cosmos Club.

When Mr. Green had completed a half century of service, on March 7th, 1913, some of his friends, all well-known men, gave expression to their high regard for his abilities and character in a signed testimonial which may fittingly close this brief sketch of his career:

"On this day you will have completed fifty years of service to the United States. To few indeed has such a term been vouchsafed, and, while the individual qualities which have characterized your own relation with the government are happily not unique in its history, few men indeed, can show a record which combines qualities so useful in a union so complete of inventive skill, technical knowledge, executive energy, simplicity in method, firmness without bluster, persistent and unselfish devotion, inflexible integrity, and a modesty more than content to be anonymous.

"The permanent memorials of these qualities are in stone and marble. They are in structures of beauty, of dignity, and of efficiency, with which your name will forever be associated. But the friends of yourself and of the public service whose names are signed below, usurping a privilege in which scores of others would have been glad to join, desire to offer you their acknowledgments, their congratulations, and their warmest wishes, and to couple with these a memento of the day which will, they hope, help to divert such leisure hours as you may allow yourself hereafter."

Mr. Green was elected a Member of the American Society of Civil Engineers on October 2d, 1889. He served as a Director for three years (1894-95-96) and as Vice-President for two years (1906-07).

CARL ROBERT GRIMM, M. Am. Soc. C. E.*

DIED FEBRUARY 15TH, 1916.

Carl Robert Grimm was born on May 3d, 1849, at Neuwied-am-Rhein, Germany. He was graduated from the Royal Prussian Gewerbeschule in Coblenz, in 1869. After serving in the Franco-German War of 1870-71, he entered the Polytechnic School at Aachen, and finished his studies in 1875 at the University of Berlin.

After one year as a Locomotive Draftsman, Mr. Grimm was engaged, in the service of the German Government, at Strassburg, in working out designs for plate girders and in experimenting on building materials for railroads.

In 1881, Mr. Grimm came to the United States and began his career as a Bridge Engineer as Draftsman with the Phœnix Bridge Company at Phœnixville, Pa. He was afterward employed with the Dominion Bridge Company in Montreal, Que., Canada; the Wrought-Iron Bridge Company in Canton, Ohio; and the Mount Vernon Bridge Company in Mount Vernon, Ohio.

In 1890, he was appointed Engineer with the Tacony Iron and Metal Company, of Philadelphia, Pa., and while in the service of this Company designed the steel framework which caps the tower of the City Hall in Philadelphia and supports the bronze statue of William Penn.†

From 1895 to 1897, Mr. Grimm was employed, as Designing Engineer, by the New Jersey Steel and Iron Company, of Trenton, N. J., and from 1898 to 1901 by the Elmira Bridge Company, Limited, of Elmira, N. Y. As Engineer of this Company, he designed the Kinzua Viaduct on the Erie Railroad at Bradford, Pa.‡ In 1907-08, he was employed by the Pennsylvania Steel Company.

In 1908, Mr. Grimm came to New York City, where he continued to reside until 1913, when he went to Europe. During this time, in 1904 and, again, in 1908, he was in the service of Gustav Lindenthal, M. Am. Soc. C. E., whom he assisted in the design of the Hell Gate Bridge and other structures for the New York Connecting Railway. He was also engaged in literary work, his book on "Secondary Stresses in Bridge Trusses", having been published in 1908.

On his return to Europe in 1913, Mr. Grimm settled in Paris, France, where he was engaged in consulting work until 1914 when he went to Neuwied-am-Rhein, Germany, where he died after a short

* Memoir prepared by John C. Trautwine, Jr., Assoc. Am. Soc. C. E., supplemented by material on file at the Society House.

† "The Tower of the New City Hall at Philadelphia, Pa.", by C. R. Grimm, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. XXXI, p. 249.

‡ "The Kinzua Viaduct of the Erie Railroad Company", by C. R. Grimm, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. XLVI, p. 21.

illness. He is survived by his widow, who is a native of the State of Ohio.

Of Mr. Grimm, Mr. Lindenthal writes as follows:

"He had a strong sense of duty and loyalty to his work; he was exceedingly painstaking and particular in his method of work, and had a rare sense of order. He had great merit as a designing engineer, particularly of structural details, in which his ideas were often original and always carefully worked out to the last rivet.

"His work on 'Secondary Stresses in Bridge Trusses' is witness of his painstaking analysis of theoretical niceties and of his great usefulness in modern designing.

"He had been a most valuable assistant to me, and I was sorry when he made up his mind to withdraw from regular office work and give more attention to his personal affairs."

Besides his papers on the City Hall Tower at Philadelphia, Pa., and the Kinzua Viaduct, Mr. Grimm also contributed one entitled "The Arch Principle in Engineering and Esthetic Aspects, and Its Application to Long Spans",* which was presented before the Society on November 2d, 1910.

Mr. Grimm was elected a Member of the American Society of Civil Engineers on June 4th, 1890.

* *Transactions*, Am. Soc. C. E., Vol. LXXI, p. 233.

EDWARD MACAULAY HARTRICK, M. Am. Soc. C. E.*

DIED AUGUST 17TH, 1915.

Edward Macaulay Hartrick was born in Belfast, Ireland, on January 11th, 1847. In his youth he suffered from acute asthma, and could not sleep unless he was propped up in bed. His doctors having ordered a sea voyage, he went as midshipman in the Green Company (British East India Company) service from London. He served there about three years, and returned home completely cured.

He was educated and received his degree in Dublin University, being graduated second in a class of twenty-nine and a special, in 1868. He was engaged in the Engineering Department of the Midland Great Western Railway of Ireland at some time either before or after graduation.

In 1869 Mr. Hartrick came to the United States and joined his brother at Fishkill Village, N. Y., in work on the Connecticut Western Railway. This lasted until 1870 or 1871. He then moved to Port Jervis, thence to Pittsburgh in 1871, and to Chicago in 1872, at each of which places he was engaged on railroad work. He served during this time with the Marietta and Pittsburg Railroad; the Shephang Valley Railroad; on the Suspension Bridge over the Delaware River at Port Jervis; the Milford, Matamoras Railroad, of Pennsylvania; and the Pittsburgh, Cincinnati and St. Louis Railroad.

Some time after 1872 he moved to Texas, and was engaged on the location and construction of the Texas and Pacific Railway, and from this time forward he spent his life in that State.

From about 1880 to about 1886 Mr. Hartrick was City Engineer of Galveston, Tex. Then he was in private practice for about a year, and in 1887 entered the service of the United States Engineer Department as Sextant Observer, and was engaged in making various surveys. From that date until his death, 28 years of continuous service, he was with that Department. In 1888 he was placed in charge of the Galveston Bay Ship Channel work, now known as the Houston Ship Channel. In 1890, on the resignation of H. C. Ripley, M. Am. Soc. C. E., he was made Principal Assistant Engineer, with immediate charge of the improvement of Galveston Harbor, the most important work in that Engineer District at the time. On the completion of the present jetties at that harbor, in 1898, he continued to act as Principal Assistant Engineer on all the work in the District. In 1912, various ocular troubles rendering further office work impossible, he was transferred

* Memoir prepared by C. S. Riché, M. Am. Soc. C. E., and N. T. Blackburn, U. S. Asst. Engr., U. S. Engr. Office, Galveston, Tex.

to the charge of the Houston Ship Channel work. On August 16th and 17th, 1915, while at his post of duty on the quarter boat at Morgan Point, a severe tropical hurricane passed over that section of the Gulf Coast, the quarter boat was wrecked, and he was drowned.

Mr. Hartrick was a member of Harmony Lodge No. 6, F. & A. M., San Felipe de Austin Chapter No. 1, Royal Arch Masons, and Texas Consistory No. 1, Scottish Rite Masons, all of Galveston. On October 18th, 1905, he was elected to the 33d honorary degree. He was also a member of El Mina Temple, A. A. O. N. M. S. He was a member of the Episcopal Church.

Mr. Hartrick was married to Miss Mary McClellan, of Galveston, Tex., and survived his wife by about 13 years. They had no children.

His death was directly due to his sense of duty. Knowing that the hurricane was coming, he left a place of safety at Galveston, went past his home at Dickinson, and took his station on his boat at his official place. Here he remained until the end. Loyal and steadfast in life, in death unafraid, he has passed to his reward.

Mr. Hartrick was elected a Member of the American Society of Civil Engineers on February 1st, 1899.

FREDERICK WILLIAM DOANE HOLBROOK, M. Am. Soc. C. E.*

DIED APRIL 13TH, 1916.

Frederick William Doane Holbrook, the son of Edward Ridgeway and Frances Louise (Doane) Holbrook, was born on Beacon Hill, Boston, Mass., on January 26th, 1840. He was educated in the public schools of his native city, taking a Franklin Medal at the Adams School, and the Abbott Lawrence Prizes in French and mathematics at the English High School.

Professionally self-taught in great measure, Mr. Holbrook's education was enriched and enlarged through an intense thirst after knowledge, which led him, at all times and everywhere throughout his life, to seek information beyond that applicable to or offered by the work on which he was immediately engaged. He was, therefore, never at a standstill, content with present attainments, but was reaching forward to new acquirements in order that he might the better perform the duty of to-day and be prepared for that to which he might be called to-morrow.

In consequence, the positions which he filled have been important, and, in each, we find him doing his work patiently, carefully, earnestly, and conscientiously, faithful always to the charge committed to his trust. The record of his services which follows, covering an active professional life of fifty-five years, is one in which he rightfully took pride.

He began his engineering work in the office of the late Thomas Doane, M. Am. Soc. C. E., a civil engineer of Boston and Charlestown, Mass., and, in 1861, became Assistant Engineer of the Old Colony Railroad, with an office at Fall River, Mass. When the Civil War broke out, he enlisted at once, and served with his regiment until, his ability coming to the notice of the Secretary of War, he was, by special order of that officer, transferred to the Engineering Department engaged in the construction of the defenses of the City of Washington.

Resuming the practice of his profession on his discharge in 1865, Mr. Holbrook was employed as Resident Engineer at the east end of the Hoosac Tunnel for two years, when he became Division Engineer of the Lebanon Springs Railroad in Vermont in 1867, of the Burlington and Missouri Railroad in Iowa in 1868, and Assistant Chief Engineer and Superintendent of the latter road in 1869, continuing as such until 1874, when he was appointed City Engineer of Plattsmouth, Nebr.

Recalled to the East, he served as Division Engineer of the Troy, Greenfield and Hoosac Tunnel Line, with an office at Shelburne Falls, Mass., in 1875-76. He was then appointed Assistant Engineer of the

* Memoir prepared by Frank O. Maxson, M. Am. Soc. C. E.

Boston Water-Works, at Framingham, Mass., a position which he filled from 1876 to 1879.

Returning to the West, Mr. Holbrook was employed from 1879 to 1888 on the Northern Pacific Railroad, as follows: as Principal Assistant Engineer, at Mandan, N. Dak., in 1879-80; as Locating Engineer, 1880-81; as Division Engineer, Wisconsin Division, Superior, Wis., 1881-84; as Lease Agent, St. Paul, Minn., 1884-86; and as Division Superintendent, Yellowstone Division, Glendive, Mont., 1886-88.

The farther West called him, however, and, in 1888-89, he was employed as Principal Assistant of the Seattle, Lake Shore and Eastern Railway and S. E. Construction Co., at Seattle, Wash., becoming Manager of the railroad in 1889-90. Afterward, he served for two years as Secretary of the Board of Public Works of the City of Seattle, Wash.

On December 5th, 1892, Mr. Holbrook entered the Government Service at the Puget Sound Naval Station as Principal Civilian Assistant, and continued his work as such during the ensuing twenty-three years, during which time the Station developed from a wilderness to a fully equipped Navy Yard, with piers, dry docks, shops, water-works, and all other appliances, second to none. From 1898 to 1903, he was in charge of all engineering work at the Station, and from 1899 to 1901 was Acting Head of Department, and, as such, planned and constructed the improvements of those years.

During the later years of his life, Mr. Holbrook's health seriously declined, mainly through an accident received while on duty at the Navy Yard, but his indomitable will and nervous energy kept him steadily at work, performing his duty as if there was no such thing as failing bodily strength. Retaining in full power his mental vigor, he labored with all the push of youth in the accomplishment of work given him to do, and it was only a few weeks before his death that he felt compelled to resign the position he had filled so long and so acceptably, because he could no longer make his enfeebled body obey his will.

The writer's intercourse with Mr. Holbrook during the time that they were together at the Puget Sound Naval Station—somewhat more than a year—was rather that of friend with friend than of principal and subordinate. He was an older man than the writer, with broader experience and knowledge, and was unfailing in devotion to duty, ever ready to undertake and perform, capable, intelligent, and faithful. Given something to do, he did it, without watching, without urging. He made each job his own, to be done in the best way, in the least time, and at the least expense, considering the end in view in its performance. It is no wonder then that he was relied on, trusted, and believed in by every one with whom he had to do, whether employer

or employed. From their first meeting the writer can recall no time when there was a veil of formality between Mr. Holbrook and himself. The work proposed, planned or ordered, was talked over as those discuss subjects in which they are mutually interested, and so the friendly official association soon extended to personal matters, and there grew that happy sense of fellowship which the passing years left unbroken. The writer feels assured that his experience has been that of every one who knew Mr. Holbrook; that everybody who met him, in public or in private, found him a Man.

On Mr. Holbrook's seventy-sixth birthday, on January 26th, 1916, which was also the birthday of his wife, the office force of the Puget Sound Navy Yard, Department of Public Works, in which he had been continuously employed since December 5th, 1892, until his resignation a few days previous, on account of ill-health, sent him a large box of beautiful flowers, and with it a letter signed by officers and men alike, embodying a very flattering address concerning his services in the past, and their best wishes for his future happiness and health. This genuine expression of good will and appreciation touched Mr. Holbrook's heart, and, in a letter to the writer, after reference to this incident, and after stating his endeavor always to assist all who came to him, he continued:

"I recognize that the only permanent satisfaction obtained in life is that due to helping others even in small matters, and also how little it takes to gratify both those of tender years, and the aged, who are so often neglected."

And this satisfaction Mr. Holbrook deserved and received.

Mr. Holbrook was a charter member of the Pacific Northwest Society of Engineers; a member of John F. Miller Post, G. A. R.; of St. John's Lodge, No. 9, F. and A. M.; of Seattle Chapter No. 3, Royal Arch Masons; and of Seattle Commandery, Knights Templar. He belonged to the Old School Boys Association of Boston, Mass., and was eligible for membership in the Sons of the American Revolution. His funeral was held under the auspices of the Masonic Fraternity.

He was married to Miss Nellie J. Barker, at Medford, Mass., on August 14th, 1862, and is survived by his wife, two sons, Paul and Fred. P., and by a daughter, Jennie, all of Seattle, Wash.

Mr. Holbrook was elected a Member of the American Society of Civil Engineers on October 6th, 1886.

WILLIAM EDWIN HOYT, M. Am. Soc. C. E.*

DIED APRIL 2D, 1916.

William Edwin Hoyt, son of Alfred Metcalf and Harriet Fabyan Hoyt, was born in Portsmouth, N. H., on July 3d, 1845. He was fitted for college at Phillips Exeter Academy, and was admitted in 1865. Becoming dissatisfied with a strictly classical collegiate course, however, he left at the end of his Sophomore year and entered the Massachusetts Institute of Technology, where he continued his engineering studies through the regular course, and was graduated in June, 1868, with the degree of Bachelor of Science in Civil Engineering.

Mr. Hoyt had his first experience in professional work several years before he began his studies at the Institute. During his summer vacations, while at the Academy and in college, he was employed on the Government fortifications which were then building at the entrance of Portsmouth Harbor, his father having charge of this work under Col. J. N. McComb, of the United States Army Engineer Corps. For the protection of the harbor and adjoining coast, three forts were built, Fort Constitution, Fort McClary, and another extensive earthwork on the east side of the Piscataqua River, and the experience gained in this work of massive construction was of great value to the young engineer.

Mr. Hoyt's connection with railroad work began in 1868, on the Chicago, Burlington and Quincy Railroad, where he was employed for a considerable time in making surveys for branch lines. Afterward, he was sent to take charge of building an important extension in Iowa. On its completion he returned to Illinois and built two divisions of the Dixon, Peoria and Hannibal Railroad, a subsidiary of the Burlington. Mr. Hoyt was then engaged for some time in the examination and inspection of new railroads which had been built for the Chicago, Burlington and Quincy Railroad in Illinois and Iowa.

In 1873, at the time when activity in railroad building had in some degree abated, Mr. Hoyt went to Europe in order to familiarize himself with English and Continental engineering practice and study the best examples of engineering work abroad. Considerable time was spent in England, Austria, Switzerland, Italy, and France, especially in the mountainous districts, where railroad building had been attended with unusual difficulties on account of great natural obstacles.

After gaining valuable knowledge and experience in this manner, he returned to the United States and established himself as a Civil and Consulting Engineer in Boston, Mass. He soon received an

* Memoir prepared by George W. Kittredge, M. Am. Soc. C. E., and others connected with the New York Central Railroad Company.

offer from the Massachusetts Institute of Technology to devote part of his time to the instruction of students of the Engineering School, and the Departments of Field Engineering and of Bridge and Roof Construction were put into his hands. In addition to the regular outside work of his office, Mr. Hoyt superintended these Departments of the Institute for several years, until other matters demanded his attention away from Boston, and he was obliged to devote himself to more active occupation.

In 1880, Mr. Hoyt was employed to inspect railroads in Canada in the interest of Eastern capitalists, and, subsequently, he entered the services of the Lake Erie and Western Railroad as Locating Engineer of its contemplated branch from the main line to St. Louis, Mo.

Early in the spring of 1881, before the Lake Erie and Western surveys were completed, Mr. Hoyt was appointed Chief Engineer of the Buffalo, Rochester and Pittsburgh Railway Company, in charge of construction and maintenance, and this position he held for almost twenty years. Under his administration, important extensions of the road in New York State and in Pennsylvania were surveyed and built, increasing its length more than threefold, with a corresponding gain in carrying capacity and earning power.

In 1900 Mr. Hoyt entered the service of the New York Central and Hudson River Railroad Company to take important assignments in the Engineering Department.

In 1901, he was sent by the National City Bank to Mexico to study the railway systems and make a detailed report on their condition, their capacity, and their prospects.

In 1905, he was sent to the International Railway Congress in Europe as an official representative of the New York Central Lines, and made an extended contribution to the discussion of that body.

During Mr. Hoyt's connection with the New York Central Railroad in his later years, he occupied a unique position. His thorough knowledge of railroad affairs and his broad and comprehensive treatment of his subject made him a particularly valuable expert witness for the Company in any matters involving its relations, not only with municipalities, but with State and Government commissions. His grasp of detail and his accuracy were remarkable for a man of his advanced age.

Mr. Hoyt served as Health Commissioner of Rochester, N. Y., from 1892 to 1897. He was a member of the American Railway Engineering Association, the Society of Colonial Wars, and the National Geographic Society. In Rochester, he was a member of the Genesee Valley Club, the Country Club, and the Fortnightly Club, and, in New York City, of the Transportation Club.

In 1876, Mr. Hoyt was married to Susan Rogers White, of Boston, Mass., who, with two children, Dr. C. Wentworth Hoyt and Mrs. C. Henry Mason, both of Rochester, N. Y., survives him.

Mr. Hoyt had a host of friends and few enemies. He was a man of extraordinary personality, combining a hearty geniality with an inborn sense of personal dignity. His sense of humor was a delight to his friends; and his counsel was much sought.

His eager, reverend curiosity concerning life and its meaning and possibilities, which grew in his mind with the passing years; his candor of spirit, which came to shine ever whiter and clearer as that spirit found refinement by life's experience, and truth came ever to be more loved and error to be more despised; the integrity of character which the long years of honest-mindedness and faithful work developed; and his wealth of friendliness, which accumulated with the growing knowledge of the years, justly earned for him his enviable reputation.

In every aspect of his vigorous and useful life Mr. Hoyt gave the utmost of those sterling qualities which formed his remarkable personality.

Mr. Hoyt was elected a Member of the American Society of Civil Engineers, on March 5th, 1884.

WILLIAM HUNTER, M. Am. Soc. C. E.*

DIED APRIL 2D, 1915.

William Hunter was born on May 25th, 1854, at Moselem, Berks County, Pa. He was educated at the Polytechnic College of the State of Pennsylvania, from which he was graduated in 1872 with the degree of C. E. He immediately began practising his profession by entering the service of the Philadelphia and Reading Railroad Company, and gradually rose from the position of Rodman to that of Assistant Engineer.

In 1876 he engaged in private practice in the ore mines at Moselem for about a year, when he again took up railroad work by entering the employ of the Pittsburgh and Lake Erie Railroad Company as Division Engineer. He remained with that company until the fall of 1878, when he returned to The Reading as Assistant Engineer. Later, he was advanced to Assistant Road Master, then Assistant Chief Engineer, and on August 9th, 1900, on the retirement of the late Col. H. K. Nichols, he became Chief Engineer, which position he held until his death.

While he was Chief Engineer Mr. Hunter built the Port Reading Railroad, together with the extensive coal docks at Port Reading, the Reading Belt Line Railroad, the Norristown and Main Line Connecting Railroad, the New York Short Line Railroad, the Delaware River Bridge at Yardley, the Rutherford and St. Clair Yards, the Low-Grade Freight Line at Wayne Junction, Philadelphia, and rebuilt the Philadelphia, Harrisburg and Pittsburgh Railroad.

In co-operation with the engineers of the City of Philadelphia, he also executed the work necessary to abolish grade crossings on the Philadelphia, Germantown and Norristown Railroad between Green Street and Wayne Junction, and on the Richmond Branch of the Philadelphia and Reading Railway between Somerset and Richmond Streets.

He was a member of the Engineers' Club of Philadelphia and the Franklin Institute. At the time of his death he was Vice-President of the Philadelphia Association of Members of the American Society of Civil Engineers.

Mr. Hunter was an engineer of marked ability and tireless energy, and held broad and progressive views of the railroad problems of his time. His professional attainments, sound judgment, and constructive skill admirably adapted him to meet the responsibilities of a successful career.

* Memoir prepared by George S. Webster, Edward B. Temple, and Samuel T. Wagner, Members, Am. Soc. C. E., a Committee of the Philadelphia Association of Members of the American Society of Civil Engineers.

As Chief Engineer of a great railroad system, serving many large cities, he had many problems to solve with municipal authorities, and, though he was a man of great firmness of character, he discussed proposed improvements with such an open-minded view of the rights of all parties that he won the esteem and confidence of those who had dealings with him.

His uniform courtesy made it a pleasure to be in his company, and he was possessed of qualities of mind and heart which endeared him to his friends and commanded the respect and admiration of all who came in contact with him either professionally or socially.

Mr. Hunter was elected a Member of the American Society of Civil Engineers on June 5th, 1895.

LINDSEY LOUIN JEWEL, M. Am. Soc. C. E.*

DIED SEPTEMBER 5TH, 1915.

Lindsey Louin Jewel was born at Christiansburg, Va., on November 24th, 1877, and was graduated in 1900 with the degree of B. S., from the Virginia Polytechnic Institute. In 1902, on the completion of two years' advanced work, the degree of C. E. was conferred on him by the same institution. In 1898, while a student, Mr. Jewel enlisted in the 2d Virginia Volunteers for service in the Spanish-American War, and was mustered out at the close of the War with the rank of Corporal.

Mr. Jewel's first work after graduation was with the Penn Bridge Company, Beaver Falls, Pa., and, while so employed, he designed and built extensions to the sanitary sewer system for New Brighton, Pa. In 1903, he entered the service of the McClintic-Marshall Construction Company, Pittsburgh, Pa., as Designing Engineer, and during the period, 1904 to 1906, designed all the buildings for the Youngstown Sheet and Tube Company, a number of bridges and viaducts on the Virginian Railway, including the New River Bridge, at Glen Lynn, Va., buildings for the Bethlehem Steel Company, etc. In 1906 he was promoted to the position of Manager of Erection, in entire charge of all erection operations west of Harrisburg, Pa., and, while thus engaged, planned and handled the erection of Piers 57, 58, and 59, Chelsea Section, New York City, 14th Street Viaduct, Hoboken, N. J., the Pen Horn Creek Viaduct of the Erie Railroad, New River Bridge of the Virginian Railway, the Ohio River Bridge (cantilever), of the Pittsburgh and Lake Erie Railroad, Harvard Denison Viaduct, Cleveland, Ohio, the Missouri River Bridge, Kansas City, Mo., etc.

In 1910 Mr. Jewel was sent by his Company to the Canal Zone in charge of the lock gate erection for the Panama Canal, and was thus engaged when, in the fall of 1912, he organized the Central American Construction Company, and was chosen its President and Chief Engineer. In the one year of his connection with this company—he returned to the United States in search of health in October, 1913—several large docks, the railway station at Panama, and a number of minor structures, were completed.

As a resident of the Canal Zone, Mr. Jewel was actively identified with its civil and social affairs. He was a member of the local clubs, and, in recognition of his interest in development work in the Zone, President Wilson, in 1913, appointed him United States Vice-Consul at Colon.

He died at Saranac Lake, N. Y., on September 5th, 1915, after an illness of nearly two years.

* Memoir prepared by James H. Gibboney, Chf. Chemist, Norfolk and Western Railway, Roanoke, Va.

Mr. Jewel was an active member of the National Association of Audubon Societies and the American Ornithological Union, and was the author of a number of papers on Isthmian birds, his rare collection of some 400 specimens attesting to his interest and skill as a collector.

Dr. John M. McBryde, President-Emeritus of the Virginia Polytechnic Institute, adds this beautiful tribute to Mr. Jewel's manly character and scholarship:

"In the opening days of the Session of 1896-97, while President of the Virginia Polytechnic Institute, I was greatly struck by the manly bearing of one of the new matriculates. Indeed, the energy and force of character apparent in every line of his face were such as would favorably impress any intelligent observer. Within a very short time the record made by this young man, Lindsey Louin Jewel, of Christiansburg, Virginia, fully confirmed my first favorable impressions, and before the close of the first term his high standing in his classes and efficient leadership in every healthy form of college activity won for him the approval and esteem alike of professors and students. And from year to year of his collegiate course, I was delighted to see him fully measure up to the high standard of performance exhibited at its outset, and at his graduation with distinction in 1900, I confidently expected for him an exceptionally successful career in active life. Although we met but once or twice after his graduation, I was not disappointed in my expectations, for rising rapidly from year to year in his chosen profession he had, before reaching middle life, won for himself in the short period of time allowed him on earth, a position of wide and commanding influence and placed to his credit a surprising amount of engineering work of the highest grade and value.

"His removal from the scene of his activities at the very time when his promise of abounding usefulness was greatest was a loss not only to his family and section, but to the whole country as well. He left behind him, however, an example of preparation and performance of which his family, his Alma Mater, and his State may well be proud."

Mr. Jewel was elected an Associate Member of the American Society of Civil Engineers on April 4th, 1906, and a Member on November 1st, 1910.

WILLIAM CORNELL JEWETT, M. Am. Soc. C. E.*

DIED MAY 2D, 1915.

William Cornell Jewett was born in San Francisco, Cal., on December 16th, 1853. He was the son of William Cornell Jewett, of New York, and Almira Guion, the daughter of a prominent citizen of Cincinnati, Ohio. His ancestry, originally of French Huguenot, English, and Dutch extraction, was purely American for more than 200 years.

At the age of sixteen, he was sent from California to Cincinnati to continue his education, and, in due time, he was graduated from the Chickering Institute, at that time among the most noted polytechnic schools in the Ohio Valley.

Soon after his graduation, Mr. Jewett was employed in the office of Mr. S. W. Hartwell, a civil engineer in Cincinnati. In June, 1873, he joined the construction force of the Cincinnati Southern Railroad, beginning as Chainman and advancing by promotion to the position of Clerk and Office Assistant to the late George B. Nicholson, M. Am. Soc. C. E., Division Engineer, continuing in this position until about the end of 1877, except for a few weeks' service as Chainman and Rodman on the surveys for the Covington and Pound Gap Railroad.

During a part of 1881, Mr. Jewett was Resident Engineer's Assistant on the Richmond and Alleghany Railroad. Following this, he was Resident Engineer on the Toledo, Delphos and Burlington Railroad, at Wellston, Ohio, in charge of 13 miles of construction work.

From January, 1882, to August, 1883, he was Resident Engineer on the New Orleans and North Eastern Railroad in charge of 14 miles of construction and 30 miles of track-laying, under Mr. Nicholson, Division Engineer.

After a few months' service as Assistant in the office of the Division Engineer, Northern Division, he was transferred to the general office of the Cincinnati, New Orleans and Texas Pacific Railroad, at Cincinnati, Ohio, where under the direction of the late L. G. F. Bouscaren, M. Am. Soc. C. E., Chief and Consulting Engineer of the System, he made a study of railroad ferry transfers, and designed the transfer ferry across the Mississippi River at Vicksburg, Miss., on the Vicksburg, Shreveport and Pacific Division. In August, 1885, he was advanced to the position of Resident Engineer of the Cincinnati, New Orleans and Texas Pacific Railroad, between Cincinnati, Ohio, and Birmingham, Ala., having charge of lining tunnels, building shops, laying out yards, and other construction work.

In August, 1893, Mr. Jewett was appointed Chief Engineer of the 30-mile extension into Cleveland of the Cleveland, Lorain and Wheeling

* Memoir prepared by S. Whinery, M. Am. Soc. C. E.

Railroad, and remained in charge of the location and construction of this important work until its completion.

Returning to Cincinnati in 1897, he was engaged for 12 years on the construction of the new water-works for that city, one of the largest and most important water-supply projects of the time, of which Mr. Bouscaren was Chief Engineer until his death, when he was succeeded by George H. Benzenberg, Past-President, Am. Soc. C. E.

The preliminary and location work on the great settling reservoirs at California, Ohio, was done by Mr. Jewett, after which, as Resident Engineer, he had charge of the construction of these reservoirs and the completion, after the death of Alfred Petry, M. Am. Soc. C. E., of all the work in that vicinity, including the coagulating basins, the construction of the river pumping station and coal-storage buildings, the general supervision of the erection of the four 30 000 000-gal. pumping engines, the laying of the mains connecting these pumps with the settling reservoirs, the intake on the Kentucky shore, and grading the grounds, building roads, and other work. Some idea of the importance of this work may be obtained from the fact that the great settling basins cover 80 acres and have a capacity of 330 000 000 gal.

Of Mr. Jewett's services on this work, an official connected therewith writes:

"In all this work he proved himself to be a valuable assistant, as he was exceedingly conscientious in the performance of every duty assigned to him, and exacted from every Contractor the full performance of all contract requirements, while his reports were always precise and complete."

His last important professional engagement was that of Chief Engineer of Construction of the new Cincinnati General Hospital. This is one of the largest and most carefully designed and constructed for its purposes among modern hospitals, comprising eighteen large buildings, with every modern improvement that medical science and experience could suggest. The cost was approximately $4 000 000.

On the completion of this work, Mr. Jewett took a well-earned and much-needed vacation, going to his native State, California, where his mother and brothers still lived at Santa Paula, and where, in the midst of beautiful scenery and surroundings, sitting in the shade of a grand live oak, he was attacked by heart failure and expired.

On hearing of his death, the Commissioners of the Cincinnati General Hospital met and adopted the following resolution:

"The sudden death of Wm. C. Jewett on May 2d has brought deep sorrow to this commission whom he represented as Chief Resident Engineer throughout the whole period of construction of the Cincinnati General Hospital.

"In recognition of his splendid and faithful services to the City and this Board be it:

"*Resolved*: That we, the Board of Hospital Commissioners, having by constant and intimate association learned to respect and deeply appreciate the high efficiency, incorruptible and sterling qualities of Mr. Jewett, desire to pay this last tribute to his memory, and to point to him as an excellent example of the ideal public servant. * * *"

This is only one example of the high regard and confidence in which Mr. Jewett was uniformly held by his employers and superiors.

His professional work was characterized by competency, sound judgment, painstaking care and accuracy, energy, and entire devotion to his duties. He was one of those engineers who assumed that contracts and specifications mean what they clearly state, that their requirements are intended to be complied with, and that it is the right and duty of the engineer to enforce them, strictly, but justly. Accustomed, himself, to scrupulous compliance with all his engagements, he expected something of the same fidelity from others.

Personally, he was a man of the highest integrity and honor, modest and quiet-mannered almost to a fault. A constant student, fond of scientific pursuits, he was most at home with his family and his books, but was not neglectful of the duties of the good citizen.

Mr. Jewett was married in 1878, to Ella Gibson, the daughter of John B. Gibson, of Cincinnati, who, with a daughter and son, the latter engaged in engineering, survives him.

Summing up the professional life and work of Mr. Jewett, it may be said that he belonged to that large class of engineers whose work honors the Profession, but who shun rather than court popular or professional credit and distinction, finding their sufficient reward in the consciousness of duties courageously met and ably and faithfully performed.

He had been a member, since 1890, of the Cincinnati Engineers' Club, of which he was a Director in 1898, Vice-President in 1900, and President in 1901. He also belonged to The American Association for the Advancement of Science and the Natural History Society of Cincinnati.

Mr. Jewett was elected a Member of the American Society of Civil Engineers on June 3d, 1885.

JOHN HOWARD JOHNSTON, M. Am. Soc. C. E.*

DIED MAY 8TH, 1913.

John Howard Johnston died suddenly in Lima, Peru, on May 8th, 1913, at the age of about 63. Born in America, he served, as a boy, in the Civil War. Subsequently, he entered the Scientific Department of Dartmouth College, from which he was graduated in the Class of 1870.

In September, 1870, he was engaged on the construction of the Westfield and Holyoke Railroad, of Massachusetts, and afterward served as Draftsman with the Hartford and Saybrook, or Connecticut Valley, Railroad.

In March, 1871, Mr. Johnston went to Peru, where he was employed on the location and construction of several railroads, among which were the Arequipa and Puno Railway, the Jaliaca and Cuzco Railway, the Lima and Oroya Railway, etc.

A few years after his arrival in Peru, Mr. Johnston entered into partnership with the late Mr. Jacob Backus and started the Backus and Johnston Brewery at Lima. This business proved a great success, and about 1890 was sold to an English Company.

Being full of energy, Mr. Johnston was not satisfied to retire from business, and he and Mr. Backus began mining and concentrating at Casapalca, Peru, about 14 000 ft. above sea level. At that time, as the railroad did not reach Casapalca, all the machinery for the plant had to be transported by mule. It was work of this arduous kind which especially appealed to Mr. Johnston.

After six years, having developed his mining and smelting interests there into a profitable business in which he still retained his interest, he left Casapalca and retired to Southern France. Automobiling was then in its infancy, and motoring became Mr. Johnston's principal hobby. He was the inventor of the Xenia carbureter.

Mr. Johnston spent several years between Paris and his villa at Cimiez, but, in 1909, his restlessness took him again to Peru. He constructed at Casapalca a modern smelter which now produces 8 000 tons of copper and 3 000 000 oz. of silver annually. He became so interested in his work there that for three years he never left the place. At his age, the high altitude eventually undermined even his iron constitution, for, after an apparently slight illness, he went down to Lima where he died on May 8th, 1913, quite suddenly, in his sleep.

Mr. Johnston's energy and genial manner endeared him to all who came in contact with him, and few foreigners in Peru have been more respected and admired by the Peruvians.

* Memoir prepared by Henry Cachard, Esq., Paris, France.

He introduced two industries in Peru—brewing and smelting—and, in spite of the natural difficulties of the country, he made a great success of both.

Mr. Johnston was elected a Member of the American Society of Civil Engineers on March 1st, 1876.

WILLIAM HARLIN KENNEDY, M. Am. Soc. C. E.*

DIED APRIL 14TH, 1915.

William Harlin Kennedy was born at Baltimore, Md., on June 15th, 1835. He was of Scotch-Irish ancestry of excellent record, who came to America only three generations back. His choice of a profession was not influenced by early association or environment, business interests mostly having claimed his immediate forbears. He began his work very young and almost in the infancy of railroad building, but showed such marked ability for location that, later, he took a high standing among his confreres in the carrying out of important projects.

Mr. Kennedy received his education at the Western University of Pennsylvania. On leaving that institution he at once began practical railway work, joining the surveying party of Mr. Sigismund Low on the Pittsburgh and Connellsville Railway in 1853.

In the beginning of 1857 he resigned to take the position of Assistant Engineer on a railway project in Kentucky, but financial stringency stopped the work. Another line, the Milwaukee and Beloit Railroad, gave him the responsible position of Division Engineer, but this road shared the fate of the previous one after about half a year of construction. He now took up what, in another profession, might be termed "free-lance work" and went farther West. At the beginning of 1858 he reached Kansas City, a river town with several thousand inhabitants but with little promise of its later astonishing growth. For several months he served as Assistant City Engineer under Mr. De Ham, who had been with him on the Pittsburgh and Connellsville work, and who was his valued friend. For a time Mr. Kennedy was also engaged in running meridian and standard parallel lines for the Government, and was employed by the Kansas City and Cameron Railroad.

A reply to a letter received from his brother, James K. Kennedy, who lived in Walla Walla, during the latter's return trip from a visit to the old home in Pittsburgh, in 1871, gives some idea of the Kansas of ante-bellum days:

"Salina, the place you write from, is very familiar to me. I have shot buffalo there in 1858-59. It was laid out as a town site by a Scotchman, a reporter for the New York *Tribune* during the Border war—I have forgotten his name. When I was surveying in that vicinity I got all the inhabitants of Salina, one named Spielman, a brother-in-law of the proprietor, and the other a little Scotchman who had taken up a claim adjoining the town site, to go with me as chainmen, and they worked the whole season, leaving the town of Salina entirely

* Memoir prepared by the late Virgil G. Bogue, M. Am. Soc. C. E.

deserted. The proprietor of Salina is the man I was telling you about when you were here, who, when attempting to catch a buffalo calf, was tossed up in the air by it and had the stock of his rifle broken off short in the fall.

"Around this town in those days and all around the plains adjoining and west and along Solomon's Fork and Grand Salina River were the hunting grounds of the Indians. Buffalo and antelope abounded in great numbers, especially the buffalo, and I have often seen the plains, as far as the eye could reach, black with them. The Indians came here in considerable numbers during the hunting season to get buffalo robes and to lay in a supply of dried meat. I believe the Pawnees claimed the grounds thereabouts, but the Kaws, the Ottawas, Kiowas, Delawares, Shawnees, Pottawattamies, Wyandottes, and a number of other tribes all hunted there, and of course some of them would fight when they met. I remember a time when we were in camp near a Pawnee camp about 40 miles beyond Salina, seeing them dance the scalp-dance around some posts which they had erected and on which were hung three scalps, some fingers, toes, and other fragments of the bodies of some Kaws whom they had that day killed in a fight. They were horribly painted and yelled like devils, and I thought at the time what a blessing it would be to the whites if all the Indians on the plains were to fight like the Kilkenny cats and with the same result."

Mr. Kennedy joined the Masonic Order while in Kansas, and took the Master Degree in Kansas City, on October 6th, 1860.

He returned to Pittsburgh in 1861 and went with the Pennsylvania Railroad as Assistant Engineer-in-Charge of Construction near Cresson. The late Alexander J. Cassatt, afterward President of the Pennsylvania System, was Rodman for Mr. Kennedy in 1861. The latter had charge of the construction of the old Union Station in Pittsburgh, which was built in 1865 and destroyed by fire during the riots of 1877. It is a peculiar circumstance that he was on the first passenger train that left the old station on its completion, and, returning from Fredericksburg, Va., was also on the last regular passenger train that ever entered it.

Mr. Kennedy was with the Pennsylvania Railroad at the time Gen. Morgan made his spectacular raid into Pennsylvania, during the Civil War. There were only a few soldiers in Pittsburgh, and the civilians were called on to aid in preparing its defenses. Rifle-pits were constructed to command the probable points of Morgan's entrance into the city. Trenches were dug on Squirrel Hill, Herron Hill, and at other points of vantage, and Mr. Kennedy was active in locating, superintending, and digging the rifle-pits around Herron Hill. Traces of these pits are still discernible there.

During Mr. Kennedy's connection with the Pennsylvania Railroad, he planned and had charge of the building of the old East Liberty stock yards, the first yard of its design to be built in the East.

In June, 1865, Mr. Kennedy resigned from the Pennsylvania Railroad to accept the position of Chief Engineer of the Oil Creek Railroad. This was a short road running from Corry, Pa., to Shafer Farm. It was built in 1865, by Messrs. Streeters and Struthers to meet the demands of traffic which had sprung up in that region during the petroleum excitement, and was a very profitable investment. At this time the road was owned jointly by the Pennsylvania Railroad, the New York Central Railway, and Streeters and Struthers, one-third of the stock being held by each of the interests.

Mr. David McCargo, at that time Assistant Superintendent of the Pennsylvania Railroad, was appointed General Superintendent of the Oil Creek Railroad by Mr. Thomas A. Scott, Manager of the Pennsylvania Railroad. On accepting the appointment, Mr. McCargo resigned from the Pennsylvania Railroad, and immediately tendered the position of Chief Engineer to his friend, Mr. Kennedy. As a result of conflicting interests, these arrangements proved only temporary, but, during the following year the whole project passed into the control of the Pennsylvania Railroad, of which it is still a part.

Mr. Kennedy then became Chief Engineer and Superintendent of the Laurel Hill and Sand Point Railroad, with headquarters at Parkersburg, W. Va., where he remained until late in 1866.

In 1867, and until February, 1868, when the work was suspended for lack of appropriations, he served as Assistant Engineer under Sigismund Low, United States Assistant Engineer, on the survey and mapping of the Ohio River, between Portsmouth and Cincinnati, Ohio, being part of the general work of which the late W. Milnor Roberts, Past-President, Am. Soc. C. E., was the Engineer in Charge, as Superintending Engineer, Ohio River Improvements.

In 1868, under the Consolidation Act passed by the Pennsylvania Legislature, the Borough of Lawrenceville, together with the Townships of Peebles, Liberty, Collins, Oakland, and a portion of Pitt, were annexed and incorporated with Pittsburgh, then occupying a comparatively small area. With the annexation of these townships, the original area of the city was greatly augmented and enlarged, and in the enlargement was included the present East End, East Liberty, Homewood, and also the Villages of Frankstown, Hazlewood, Glenwood, and others.

In order to provide for a proper system of intercommunication, a department termed the City District was organized, the surveys for which were in charge of a District Engineer, to which position Mr. Kennedy was appointed in April, 1868. His principal work comprised a comprehensive system of city planning, locating the lines of main and auxiliary thoroughfares, to connect properly the streets of the old city with those of the outlying villages and other portions of the new

sections of the city, the adjustment of the new streets for proper drainage and sewerage, as well as proper grades. This city planning entailed an enormous quantity of work, and was performed most conscientiously, all the engineering work being under his personal direction and supervision even to minute details, which in this case resulted in a perfect system of streets especially adapted to the varying topography with which the annexed district abounded.

About this time, the subject of a new water supply for the enlarged city was under advisement, and some surveys and studies had been made which finally culminated, on August 28th, 1871, in the appointment of a Board of Water Commissioners to supervise the construction of the new water-works system.

Mr. Kennedy then resigned his position as District Engineer in Charge of City Surveys of the new city district, and in the fall of 1871 was appointed Chief Engineer of the Pittsburgh New Water-Works.

The various sites for reservoirs which had been proposed, as well as others, were at once thoroughly investigated, one of the latter being at Verona, on the Allegheny Valley Railroad, about 8 miles north of Pittsburgh, on the south bank of the Allegheny River. The question of using the Allegheny River as a source of supply, or that of the Monongahela River, gave rise to much discussion, as each river had many adherents. In the case of the Monongahela, the water would be pumped into a reservoir in what is now Schenley Park. If the Allegheny was selected, the Hiland Avenue, Brilliant Hill, and Herron Hill reservoir sites would be used. Mr. Kennedy decided in favor of taking water from the Allegheny, and his decision of 45 years ago has been abundantly justified. During the winter of 1871-72, the reservoir sites of Hiland Avenue, Brilliant Hill, and Herron Hill were finally selected, and in the spring of 1872 detailed surveys were made and the work was prepared for contract. The Hiland Avenue Reservoir is the largest of the system and Herron Hill the smallest. Brilliant Hill was never fully completed, and is now known as Carnegie Lake in Highland Park, originally spelled Hiland, but later changed to Highland.*

In addition to the supervision and construction of the three reservoirs, Mr. Kennedy also had charge of the pipe distribution system, which included the large supply mains in Butler Street and Hiland Avenue, as well as the distribution system in other streets, prominently that in Second Avenue, as well as in many streets in East Liberty, generally known as the East End.

Two of Mr. Kennedy's assistants are yet living, namely Emile Low and James H. Harlow, both Members of the Society.

* A description of the Hiland Avenue Reservoir will be found in *Transactions*, Am. Soc. C. E., Vol. LXXX, p. 690, being a discussion by Mr. Emile Low on Concrete-Lined Reservoirs, etc.

The reservoirs mentioned are models of substantial construction, and, although 40 years old, have required little or nothing for repairs.

Mr. Kennedy's connection with these operations was terminated in the early part of 1874, when the work was in an advanced state of completion.

In 1875 Mr. Kennedy went to Montana as Engineer of a hydraulic mining company. During that year he rode alone from Helena, Mont., to Walla Walla, Washington Territory, over the Mullan Trail, to visit his brother, then living in Walla Walla. Thence, he went to San Francisco, arriving there at about the time the Bank of California closed its doors, on August 27th, 1875.

During the three succeeding years he was engaged in private work in Pittsburgh, and in 1879 went to the Pacific Coast and was, until 1883, Principal Assistant Engineer of the Oregon Railway and Navigation Company, under Mr. Hans Thielson, Chief Engineer, whose son, Henry B. Thielson, was Assistant Chief Engineer, and who says of Mr. Kennedy:

"He was easily the most accomplished locating engineer he [Mr. Thielson] had ever been associated with. His most important works were the line from Pendleton across the Blue Mountains to the Grand Ronde Valley, located by him and built under his direction, and the first location between Portland and The Dalles on the Columbia River. Had it been possible to build the last-mentioned line as located by Mr. Kennedy, it would have reflected great credit on the locater as well as on the company. It was a bold location, admirably balancing conditions in the equation of curvature *vs.* cost. Unfortunately, the necessity of completing a connection between the Northern Pacific at Wallula and Portland, and the impossibility of obtaining the necessary labor to build the line as located by Mr. Kennedy within the time allowed by the management, made it necessary to break up the line temporarily. However, Mr. Kennedy had an opportunity later, while Chief Engineer of the Oregon Railway and Navigation Company, of revising the constructed line largely on the lines of his original location."

The line from Celilo to Wallula on the Columbia River was also located by Mr. Kennedy, and most of it was constructed under his supervision. He had actual charge of the construction of the lines built in Oregon at that period, and later, of the lines built in Washington and Idaho, including the several branches in the so-called Palouse Country, the extension of the main line to Spokane, and the important branch in the Cœur d'Alene Mountains.

From 1884 to 1886 he was Assistant Engineer of the Northern Pacific Railway, of which the late Gen. Adna Anderson, M. Am. Soc. C. E., was Chief Engineer and Virgil G. Bogue, M. Am. Soc. C. E., Principal Assistant Engineer, engaged along the Western slope of the

Cascade Mountains, to and across Stampede Pass, and subsequently, during 1886 and part of 1887, in the Rocky Mountains of Central Montana, through the Prickley Pear Canyon and Boulder Valley.

On June 30th, 1887, Mr. Kennedy became Chief Engineer of the Oregon Railway and Navigation Company, and remained with the road until its absorption by the Union Pacific Railway in 1890.

In 1891 he was Chief Engineer of the location of the Columbia River and Astoria Railway.

Early in 1892, a number of prominent business men of San Francisco, Cal., took up the question of building a railway from San Francisco to Salt Lake City, as a line to compete with the Central Pacific and Southern Pacific. They organized a company and proceeded to discuss possible routes. Mr. Virgil G. Bogue, being at that time in San Francisco, suggested Mr. Kennedy as an engineer fully qualified for the undertaking. Mr. Kennedy, therefore, early in March, 1892, took up this work. As the outcome of his preliminary studies, he recommended investigations and surveys of several passes through the Sierra Nevada and Tehachapi Mountains. These reconnaissances were made by engineers selected by him and under his direction, but that of Beckwourth Pass he himself carried through. He also went over the proposed route from San Francisco *via* Stockton to Bakersfield, leading to passes of the Tehachapi Range.

Then followed preliminary surveys and estimates of the Beckwourth Pass Route, Mr. John T. Williams having charge between Oakland and Oroville, and J. Q. Jamieson, M. Am. Soc. C. E., of the part in the Sierra Nevada Mountains between Oroville and the State line. Mr. Bogue, subsequently, after thorough studies of many passes by himself and other engineers, recommended and adopted the general route indicated by Mr. Kennedy's studies between Sacramento and Beckwourth Pass as part of the Western Pacific, applying, however, a 1% maximum grade where the surveys already mentioned had been on 1.32 and 1.35% maxima.

The route suggested by Mr. Kennedy, from San Francisco *via* Stockton to Bakersfield, was subsequently utilized by the San Francisco and Great Salt Lake Railroad mentioned, except the part between San Francisco and Stockton, where another route was chosen in order to reach a terminus on the Bay at Point Richmond, instead of Oakland, which had been recommended by Mr. Kennedy.

In 1893 Mr. Kennedy was for a time engaged in Chicago on estimates of cost of a Belt Line from Waukegan to the Wisconsin State line.

Early in July, 1894, he was again appointed Chief Engineer of the Oregon Railway and Navigation Company, the property then being in charge of the Court, with Maj. E. McNeil as Receiver.

There had been a disastrous flood in the spring of that year, the Columbia River being higher than shown by any record, even traditionally, and great damage had been done to the roadbed of the railroad along the river. More than 100 miles of track, with the bridges, were destroyed, and through traffic was wholly stopped west of Umatilla, local passengers being transferred to boats and then again to rail to reach their destination. During this trying period, Mr. Kennedy worked almost continuously from July 4th to August 16th, sometimes without sleep for two or three days, and succeeded in repairing the damages caused by the flood in an incredibly short time, and at one-quarter the cost estimated by the former management.

In connection with the betterment work inaugurated soon after the late E. H. Harriman acquired control of this property, Mr. Kennedy made a study of every stream crossing, waterway, bridge, and culvert, also of every possible line change that would better the alignment or reduce the grade. The result of the surveys and estimates for these betterments were condensed into book form, and adopted by the Board of Directors in their entirety, and the work was carried out as proposed by him or his successors.

He was authorized to rebuild, in a permanent manner, hundreds of temporary structures, originally put in by him in the pioneer days, and also was privileged to eliminate curvature and reduce grades, which the heavier equipment and increased speed and traffic then demanded. This was all a labor of love for him. He delighted in working out the plans and details for these improvements, and fully appreciated the great economies of operation which would thereby result.

He remained with the company until 1905, when he was compelled to resign on account of a nervous breakdown. Mr. Harriman persuaded him to take a rest. His condition, however, did not improve to the extent of resumption of his responsibilities, and he tendered his resignation to Mr. B. A. Worthington, General Manager, on March 15th, 1905. This severance of relations was regretted by all his associates on the road.

In a letter to Mr. Harriman, written at the time he resigned, he said:

"I feel very grateful to you for your kind wishes expressed for my restoration to health which I believe is only a question of time, the trouble being more a nervous than an organic one. I have, however, already lost too much time, and, as the season advances, the work on the road will be pressing. I have, therefore, placed my resignation in the hands of General Manager Worthington."

After a long rest, with complete freedom from care, his health did improve. Late in 1905 he was associated with Mr. Bogue in New York, and became Consulting Engineer of the Western Maryland Railway. He was thus brought into active outdoor life again. This change

materially helped to restore his health, so that when Mr. Bogue began the construction of the Western Pacific Railway, Mr. Kennedy was able to take up the duties of Consulting Engineer for a part of the time during 1907-08-09.

In the spring of 1915 his death was caused by an acute attack of pneumonia which lasted only 10 days.

In many ways Mr. Kennedy was marked by Nature for the career which he followed, and he united in his character exceptional traits making for the higher life. All who knew him, especially his subordinates, found him kindly and considerate. He had a quick perception of the good and true in others, and was generous in his judgments throughout a long life of varied associations and achievements. He never married, although he was domestic in his tastes, and a man to whom children particularly appealed. Beginning his professional duties at a time when public works were less numerous than now, he yet was constantly employed, gaining experience which his ready adaptability made of the greatest benefit when he assumed the larger responsibilities of later years. He may safely be called one of the great engineers of his time and country. Too much cannot be said, both of his ability to carry to successful completion any project seriously undertaken by him, and of his single-mindedness of purpose.

Like most men of his calling, Mr. Kennedy was above cheap advertising, relying on his record and reputation for continued success.

He will long be missed by both friends and associates. To his family his death cannot be other than an irreparable loss.

Mr. Kennedy was elected a Member of the American Society of Civil Engineers on September 6th, 1871, and was elected a Director on December 3d, 1901, to fill the vacancy on the Board caused by the death of George A. Quinlan, M. Am. Soc. C. E.

FREDERIC CHARLES KUNZ, M. Am. Soc. C. E.*

DIED MAY 3D, 1916.

Frederic Charles Kunz was born in Prague, Austria, on January 19th, 1862, and completed his technical studies at the Polytechnical School in that city, from which he was graduated in 1886. He spent the next 5 years in the service of the Austrian Northern (Kaiser Ferdinand) Railroad, as Assistant Engineer on important construction work, comprising about 200 miles of new lines. Young as he was, he already showed such conspicuous ability, that he was also given charge of the construction of iron and masonry bridges.

In the summer of 1891, Mr. Kunz came to the United States and found employment as Assistant Engineer in the Bridge and Construction Department of the Pencoyd Iron Works, in Philadelphia, Pa. He remained with this Company, except for the period from 1893 to 1896, during which he was engaged in designing steel and masonry bridges in the Bureau of Surveys, Department of Public Works of Philadelphia, until it was absorbed by the American Bridge Company. He then became Assistant to the late C. C. Schneider, Past-President, Am. Soc. C. E., the Vice-President and Chief Engineer of that Company, with whom he remained until the summer of 1903.

The most important work on which Mr. Kunz was engaged during his connection with the Pencoyd Iron Works was the working up of the erection plans for the Clifton Arch Bridge for highway traffic, immediately below Niagara Falls. This bridge was designed by the late L. L. Buck, M. Am. Soc. C. E., the Pencoyd Iron Works being the contractors. Mr. Kunz had also much to do with the checking of the strain sheets and details of construction, and afterward of laying out the system of erection, which had many novel and difficult features.

Later in 1903, he was appointed by the writer, at that time Commissioner of Bridges of New York City, to the position of Consulting Engineer of the Bridge Department, where he took an active and important part in the preparation of the specifications and plans for the Manhattan Bridge and the Queensborough Bridge. Early in 1904, he resigned to become Chief Engineer of the Bridge Department of the Pennsylvania Steel Company, in Steelton, Pa., where he remained for several years.

The most prominent work of which Mr. Kunz had charge during this period was the fabrication of the Queensborough Bridge. When, in 1908, the safety of this bridge became a subject of controversy, he wrote a comprehensive report on it. During his connection with

* Memoir prepared by Gustav Lindenthal, M. Am. Soc. C. E.

the Pennsylvania Steel Company, he designed a large number and variety of railroad bridges which were fabricated in Steelton. It was at this time that he commenced to write his book on "Design of Steel Bridges", for which he had accumulated data and material for many years previously.

His last construction work was the steel arch highway bridge at St. Johns, New Brunswick, over the St. Johns River, which was built to replace an old wire suspension bridge at the same place. In the preparation of these plans, he was associated with his friend, Mr. C. C. Schneider, whom he followed in death within 4 months.

The personal and professional life of Mr. Kunz, was simple and unostentatious and although an engineer of exceptional capacity and attainments in his particular branch, steel bridge engineering, he was not as widely known as his worth deserved, because of his innate modesty. His name, however, will remain known to bridge engineers through his work on "Design of Steel Bridges", in which he perpetuated his fine analytical mind, as well as his practical experience. His book is not only a valuable gift to the bridge specialist, but an honor to the Profession at large, which esteems and appreciates engineering ability of high quality.

To those who enjoyed Mr. Kunz's intimacy and friendship, he will remain unforgotten through his charming and pleasant personality. A great fund of wit, a wide range of reading, a love of higher music, a deep sense of right, and a philosophic wisdom in human affairs, made conversation and social intercourse a privilege and pleasure to those of his friends to whom he chose to unbosom himself.

Mr. Kunz's health had been poor for several years, and it is deplorable that such a useful life as his should have been cut short by his untimely death. He left no relatives in this country, but is survived by a son and a nephew who live abroad.

Mr. Kunz was elected an Associate Member of the American Society of Civil Engineers on February 6th, 1895, and a Member on December 7th, 1898.

CHARLES ABBOTT LOCKE, M. Am. Soc. C. E.*

DIED NOVEMBER 12TH, 1915.

Charles Abbott Locke was born at La Grange, Tenn., on December 25th, 1841. Like many Southern boys in ante-bellum days, his early training was under a tutor, but he studied engineering later at the Presbyterian Synodical College in La Grange, receiving his diploma in 1861, after he had enlisted in the Southern Army as a private. His studies at college were under the personal direction of Professor J. L. Meigs.

In December, 1861, he joined the Seventh Tennessee Confederate Cavalry (Forest's Brigade). He surrendered and was paroled at Gainesville, Ala., on May 10th, 1865. He was captured near Atlanta in 1864 and taken to Winchester, Tenn., where he made his escape, crossing the mountains on foot, swimming the Tennessee River, and finally, after many hardships and narrow escapes in traversing the enemy's country, he rejoined his command. After the War, he joined his old Professor and friend, Col. Meigs, who was then Chief Engineer of the Memphis and Charleston Railroad, and served under him as Assistant on this road and also on the Memphis and Little Rock Railroad.

In 1867 Mr. Locke was married to Miss Virginia Beck, of Covington, Ga., and until 1873 was engaged in the milling business in West Tennessee.

In 1873 we find him at Memphis as Locating and Division Engineer on the Paducah and Memphis Railroad; here he had cholera, which undermined his health. He returned to Georgia and took charge of the physical survey of that State.

In 1878 he became Assistant United States Engineer in charge of improvements on the Oconee and Ocmulgee Rivers, and, later, on the Tennessee River below Florence, Ala., Muscle Shoals Canal, Cumberland and Tennessee Rivers, and their tributaries.

In 1889 he assisted in organizing the Engineering Association of the South.

In 1890, while living in Nashville, Tenn., he left the Government service and engaged in private practice.

Mr. Locke was interested in geological research, and spent much of his time in economic geological studies, especially in connection with the phosphate industries in the Southern States. Later, he also made studies and investigations for a prominent Eastern Portland Cement Manufacturing Syndicate headed by Robert W. Lesley, Assoc. Am. Soc. C. E. Mr. Locke possessed a rare instinct for this kind of work, and

* Memoir prepared by Hunter McDonald, Past-President, Am. Soc. C. E., and E. C. Lewis and W. F. Foster, Members, Am. Soc. C. E., a Committee of the Engineering Association of the South.

his careful, painstaking efforts made his reports of great value to his clients. He was fond of mechanics also, and spent much of his leisure hours in working in metals in his shop at home. He loved Nature, and preferred to study it alone, making long excursions into the wilds of Canada in search of the small-mouth bass, the fighting proclivities of which he prized very highly. Though he had had a checkered and gallant career as a soldier, he was unlike many men in that he never talked of his war record, and it was with great difficulty that even his best friends were able to draw him into a conversation regarding these adventures. Fit by birth, nature, ability, and education to fill high official positions, he served the entire four years of the war as a private soldier.

He was a man of delicate physique, but of large and strong mental and moral force. He possessed high ideals, and was the soul of honor and integrity; he was unselfishly devoted and faithful to his friends, of whom he had many; a student of public affairs, and a severe critic of the acts of public men, he contributed often to the local press, always over his own signature, and to appreciative readers. He had the courage of his convictions and principles, and asserted them through his ballot at the polls.

At the time of his death, and for many years previous thereto, Mr. Locke was an active and useful member of the Committee in charge of the Tennessee Confederate Soldiers' Home, where his knowledge of engineering was of great value.

The last act of his life was one of unselfish devotion to a friend, and his effort to shield him was the immediate cause of his death.

He died of heart failure in Nashville, Tenn., on November 12th, 1915. His wife and his daughter, Mrs. J. G. Kirkpatrick, a widow, both living at Nashville, survive him.

Mr. Locke was elected a Member of the American Society of Civil Engineers on October 3d, 1888.

THOMAS FRANCIS McCRICKETT, M. Am. Soc. C. E.*

DIED JANUARY 26TH, 1916.

Thomas Francis McCrickett, the son of Patrick and Sarah O'Flynn McCrickett, was born in Detroit, Mich., on November 9th, 1869. In early infancy his parents took him to Bay City, Mich., and it was there that he received his primary education. He was one of the first boys to be graduated from St. James School of Bay City, the pioneer parochial high school of the country, and he was as proud of his membership in the alumni of this institution as he was of any affiliation that was his. After his high school term, Mr. McCrickett spent a few years as a teacher in the country schools of Bay County, Michigan. This was an experience to which he always looked back with great pleasure, and one which he prized as among the most valuable of his life. Then came a term at Alma College, Alma, Mich., followed by his entrance into the Engineering Department of the University of Michigan. As proof of his high rank as a student and of his scholarship, two things may be related: The first is a quotation attributed to the Dean of Engineering at Ann Arbor, Professor Cooley—"Students may come and students may go, but there will never be another Tom McCrickett." The second is Mr. McCrickett's election to membership in the Tau Beta Pi Fraternity.

Mr. McCrickett's first position after leaving the University was with the Industrial Works, the leading manufacturing institution of Bay City, Mich. He remained there about one year, at the end of which period Mr. W. S. Russel, of the Russel Wheel and Foundry Company of Detroit, chose him as engineer to take charge of his growing business in the structural steel line. This choice marked the beginning of a term of service through which Mr. McCrickett won for himself a reputation as an engineer of great ability and a business man of impeccable integrity. He won likewise the esteem and admiration of his employer to the extent that the official announcement of cessation of activities at the Company's office on the day of the funeral referred to him in these words: "our beloved officer and companion". Indeed, the man and friend stood out pre-eminently in Thomas McCrickett.

Mr. McCrickett died on January 26th, 1916, at Harper Hospital, Detroit, Mich., the end coming as the result of an operation for intestinal trouble. By his death the Society has lost a worthy and distinguished member, and the Engineering Profession has been deprived of the services of a worker of mark and capacity.

* Memoir prepared by Richard H. Daly, Esq., Eng. Dept., Russel Wheel & Foundry Co., Detroit, Mich.

Mr. McCrickett at the time of his death was at the head of the Engineering Department of the Russel Wheel and Foundry Company, one of the largest fabricators of structural steel in the country. This position he had held for a number of years, and there are in the City of Detroit, throughout the State of Michigan, and in spots scattered here and there all over the land, many a quasi-imperishable monument of steel, attesting the proficiency of this engineer, whose task it was to solve the problem of what these sky scrapers, bridges, power-plants, viaducts, auditoriums, and the like should be in order to serve the purpose for which they were constructed.

Although Mr. McCrickett was an engineer of eminence, he was not engrossed in his profession to the exclusion of all else. All that was worthy in the thought or action of the community in which he lived had his ardent support. He was a zealous worker in the St. Vincent de Paul Society; he was first officer of the Detroit Assembly of the Knights of Columbus; and was an enthusiastic member of the University Club. These affiliations were as much a part of his life and labors, and are as worthy of note, as his membership in the Detroit Engineering Society, of which he was a Past-President, and his place on the faculty of the Engineering Department of the University of Detroit.

He was an engineer by profession, and merited distinction among his colleagues; yet he will live in the memory of those who knew him best as an exemplar of sterling manhood, and as a generous, trustworthy, and reliable friend.

Mr. McCrickett was elected a Member of the American Society of Civil Engineers on January 2d, 1907.

HENRY COATHUPE MAIS, M. Am. Soc. C. E.*

DIED FEBRUARY 25TH, 1916.

Henry Coathupe Mais was born at Westbury-on-Trym, near Clifton, England, in 1827. He was educated at a private school in Bath and at the Bristol and Bishop's Colleges, respectively, at Bristol, England, and completed his studies under the tutorship of Mr. John Exley, of Cambridge, England.

In 1844, Mr. Mais was articled as a pupil to Mr. William Michael Penistone, one of the Chief Engineers under Sir I. K. Brunel, during the completion of the Bristol and Exeter Railway, and, subsequently, on the surveys and construction of the Wilts, Somerset, and Weymouth Railways. On this work Mr. Mais had every opportunity of gaining a thorough knowledge and training in the principles and practice of railway location and construction. After completing his articles he went to Birmingham where he spent 18 months in the Engineering Department of the Broad Street Foundry.

In June, 1850, Mr. Mais, with the proprietor of the Broad Street Foundry, purchased a large assortment of engineering tools, patterns, and machinery, and went to Sydney, New South Wales, Australia, with the intention of establishing an engineering and manufacturing business there, but the discovery of gold on the Turon River and at other localities, almost immediately after their arrival in that country, unsettled business to such an extent that they were compelled to abandon their original intentions.

In 1851, Mr. Mais accepted the position of Engineer to the Sydney Railway Company which had been formed to construct the line from Sydney to Parramatta. He held this position for 18 months and then entered the service of the City Commissioners as one of the Assistant Engineers where he remained until 1856, when he joined the late E. D. Nicolle, in a manufacturing and general engineering business in Sydney, and carried out some extensive mechanical work, among which was the erection of the first hardwood sawmills at Ourimbah, near Gosford, New South Wales.

In June, 1858, Mr. Mais went to Victoria where he was engaged as Engineer and Manager for Cornish and Bruce, the Contractors for the construction of the Melbourne to Bendigo Railway, first on Sections 2 and 3, and, subsequently, on Sections 4, 5, and 6, which were completed in 1862.

He was then appointed Engineer and Manager of the Melbourne and Suburban, and of the Brighton Railway Companies, the lines of which extended from Melbourne to Hawthorn and Brighton Beach,

* Memoir prepared by the Secretary from material on file at the Society House.

respectively. He retained this position until June, 1866, when these companies and the Hobsons Bay Railway Company were amalgamated, and the whole system became the property of the State.

In 1865, Mr. Mais had been appointed by the Victorian Government as a member of a Board of Inquiry to investigate matters connected with building contracts for the Kew and Ararat Lunatic Asylums, and also as a member of a Board of Experts to examine into and report on the value of private wharf frontages, buildings, etc., on the north bank of the River Yarra, with the view of the purchase of these properties by the Government. He was also one of three engineers appointed by the Tasmanian Government to examine and report on the construction of the main line railway between Launceston and Hobart, which report was to enable that Government to ascertain whether the Railway Company in question had carried out its undertaking in a manner such as would entitle it to payment of interest guaranteed by the Government.

In 1866 Mr. Mais entered the Victorian Water Supply Department, where he remained until April, 1867, when he accepted the position of Engineer-in-Chief of South Australia, an office which he held continuously for 21 years until he resigned in 1888. During these years, in addition to this position, Mr. Mais held the following appointments: General Manager of Railways, including charge of the Maintenance and Locomotive Departments, from 1867 to 1879; Engineer of Water-Works from 1867 to 1878; and Engineer of Harbors from 1880 to 1888. He also constructed 1 473 miles of railway; 8 000 lin. ft. of wharves and jetties; built several first and second-order lighthouses; erected the fortifications at Glanville and Largs Bay, Port Adelaide; and had general charge of public works and roads outside District Councils, involving altogether an expenditure of £10 500 000. He had also served as a Justice of the Peace for South Australia for more than 34 years.

In December, 1882, under the direction of the Government of South Australia, Mr. Mais made a tour around the world, which extended over nine months, visiting Ceylon, Italy, Switzerland, France, Great Britain, and the United States, and returning to Sydney *via* San Francisco, Cal. On his return in 1883, he embodied his observations in a report which was published and laid before Parliament, and for which he was granted a special sum of £815.

On his retirement from the office of Engineer-in-Chief of South Australia in 1888, Mr. Mais returned to Melbourne and engaged in private practice as a Consulting Engineer and Arbitrator, in which latter capacity he acted in important disputes between railway contractors and the Governments of Victoria, New South Wales, Queensland, and Tasmania, respectively. In 1891, he was offered the position of Engineer-in-Chief of West Australia, but declined, in order to continue his consulting practice.

Among the works carried out by Mr. Mais were the following: Inspecting private railway lines in South Australia; designing and erecting a hydraulic sluicing plant for gold mining; from 1895 to 1897, Inspecting and Consulting Engineer in Victoria for extensive gold milling and mining machinery for West Australia; and from 1900 to 1902 as Consulting Engineer for the Charlotte Plains Electric Transmission Company, the New Havillah, Charlotte Plains, and Junction Deep Leads of the Victoria Gold Mines, on the erection of extensive steam and electrical machinery for power purposes. In 1902, he was appointed and served as a member of a Committee to test and report on the merits of certain locomotive spark-arresting appliances for the Victorian Government.

In 1912, after having been in active practice in Melbourne, Victoria, as a Consulting Engineer for nearly 25 years, Mr. Mais, owing to ill-health, retired to his home in South Yarra, Melbourne, where he died on February 25th, 1916.

He was elected a Member of the Society of Engineers, London, in 1873, and a Member of the Institution of Civil Engineers, London, in 1879, having held the position of Chairman of the Victorian Advisory Committee of that Society since 1890. He was also a Member of the Institution of Mechanical Engineers of London, England.

Mr. Mais was elected a Member of the American Society of Civil Engineers on June 6th, 1883.

HENRY GURNEY MORRIS, M. Am. Soc. C. E.*

DIED JANUARY 19TH, 1915.

Henry Gurney Morris was born in Philadelphia, Pa., on May 25th, 1839.

After his graduation from Haverford College, he entered the manufacturing business and, while quite young, became a member of the celebrated firm of Morris, Tasker and Company, which Company was the first to manufacture wrought-iron pipes and boiler flues.

Mr. Morris afterward became the sole owner of the Southwark Foundry, and constructed some of the largest blowing engines, pumps, and other heavy machinery used at that period, being justly considered at that time as one of Philadelphia's greatest captains of industry.

Early in his business career Mr. Morris became interested in electrical engineering and, with Mr. Pedro G. Salom, one of his intimate business associates, he invented and developed the "Electric Vehicle", for which they received the gold medal of the "Times-Herald Motorcycle Contest" in Chicago in 1895, and also the John Scott Legacy Medal of the Franklin Institute. Mr. Morris took out numerous patents in connection with this "vehicle" and also for storage batteries.

He was always a leader in new enterprises and inventions which promised to be of benefit to the country, and was one of the few men in the United States to recognize the importance and value of the Bessemer process of steel manufacture. He was also a leading designer and manufacturer of all kinds of machinery for sugar plantations and refineries, gas plants, and water-works, and early recognized the value of compound engines for use in marine engineering.

Mr. Morris was not quick to make friends, and to those who knew him only casually, he seemed to be somewhat haughty and austere; but to those to whom he gave his friendship, he was gentle, loving, and trusting as a child. He was unusually quiet and reserved in manner, but he had a keen sense of humor and a quick appreciation of merit and ability which kept him greatly in advance of his time. His general knowledge of mechanics was extraordinary, and covered nearly every branch of mechanical engineering. It has been said of him that he could build anything from a bicycle to a locomotive and from a motor-boat to a dreadnaught.

Mr. Morris is survived by his widow, who was Miss Sallie Marshall Morris, and three sons.

He was a Member and Vice-President of the American Society of Mechanical Engineers; a Member and President of the Engineers' Club of Philadelphia; a Member of the American Institute of Mining

* Memoir prepared by the Secretary from information supplied by Pedro G. Salom, Esq., Philadelphia, Pa.

Engineers, the Franklin Institute, and many other National Societies. With a single exception, he was, at the time of his death, the oldest living member of the Union League of Philadelphia, having been elected to membership in February, 1863.

Mr. Morris was elected a Member of the American Society of Civil Engineers on December 4th, 1867, and served as a Director during 1886.

OLAF RIDLEY PIHL, M. Am. Soc. C. E.*

DIED OCTOBER 14TH, 1915.

Olaf Ridley Pihl, the eldest son of Carl Abraham and Catherine (Ridley) Pihl, was born in Christiania, Norway, on January 9th, 1855. His father was the pioneer railroad engineer of Norway, and had received his first instruction in England. When the Norwegian Government built its first railroad, Mr. Robert Stephenson, a son of the more celebrated George Stephenson, was in charge of this work, and the elder Mr. Pihl became one of his principal assistants. When the work was completed, Mr. Pihl became Chief Engineer and thereafter spent almost his entire life in the railroad service of the Norwegian Government. He became known as the father of the narrow-gauge railroad, and was frequently consulted as an authority on that system, which, at one time, had many advocates who considered the 3-ft. or 3½-ft. gauge as the most suitable and economical under many conditions. In the Seventies, he was invited to Canada as the guest of honor at the opening of several narrow-gauge railroads radiating northward from Toronto.

His uncle, Oluf Pihl, was also one of the leading civil engineers of Norway, having been engaged mainly in municipal engineering, and was the first to introduce gas-works in that country. A younger brother, Einar Pihl, came to the United States in 1879, and served in the United States Engineer Office at Portland, Ore., on the Columbia River jetties. He died of typhoid fever in Portland, in 1886. The celebrated Antarctic explorer, Einar Borchgrevink, the first man to winter on the Antarctic Continent, was a cousin of Olaf Ridley Pihl, on his mother's side.

The home of Olaf Ridley Pihl's father in Christiania was one of the most enlightened and interesting in the capital of Norway. It was a great meeting place for the leading and aggressive men of science and the developing Profession of Engineering, who would gather about the fireplace of the library on winter evenings discussing and listening to live questions propounded and debated by the leading spirits of the country, as well as by men of prominence of other countries, who were fortunate in being guests at this hospitable home. The elder Mr. Pihl was a member of scientific societies and received many distinctions, such as orders of knighthood, from rulers of various countries.

Olaf Ridley Pihl's mother, Catherine Ridley, a refined and accomplished English lady, was a lineal descendant of Bishop Nicolas Ridley.

Mr. Pihl's boyhood was spent in Christiania, and he received his

* Memoir prepared by Emile Low, M. Am. Soc. C. E.

technical education at Chalmers Polytechnical College (Chalmarska Institut), in Gothenburg, Sweden, from which he was graduated in 1876 with the degree of civil and mechanical engineer.

During his college years and the four years after his graduation—from 1876 to 1880—he was employed in the Norwegian Government service as Second and afterward as First Assistant Engineer on the survey of the Western Railroad, from Christiania to Laurvik.

Due to his father's Canadian trip, Mr. Pihl chose Canada as a field of action, and, provided with letters of introduction to his father's friends, left Norway in 1880 for Toronto, Ont., Canada, in company with two of his college classmates, Mr. A. L. Hertzberg, now Division Engineer of the Canadian Pacific Railroad, and S. M. Kielland, M. Am. Soc. C. E., now Chief Engineer of the Buffalo Creek Railroad, and Norwegian Consul at Buffalo, N. Y.

The party, however, did not stop in Canada, but proceeded to Portland, Ore., where Mr. Pihl secured service with the Oregon Railway and Navigation Company as Topographer and, later, as Resident Engineer on construction, remaining with this railroad from 1880 to 1884. From 1884 to 1887, he was Engineer for Hoffman and Bates, Bridge Builders, of Portland, Ore.

In 1887, he entered the service of the Federal Government in the United States Engineer Office, at Portland, Ore., as Assistant Engineer in charge of the construction of the canal and locks at the Cascades of the Columbia River in Oregon. While in this service, he designed a boat railway scheme for overcoming the obstructions in the Columbia River between The Dalles and Celilo Falls, for which a canal and locks were finally adopted. He was also engaged on the jetty work at Yaquina Bay, and made several surveys and examinations for work in the Portland District. In 1898 he severed his connection with that office and removed to Buffalo, N. Y. During this period, he was engaged in contracting work for a short time.

After his arrival in Buffalo, Mr. Pihl received an appointment as U. S. Assistant Engineer from Major (now Colonel) T. W. Symons, Corps of Engineers, U. S. A. (Retired), in charge of the reconstruction of a portion of the Buffalo Breakwater.

In 1900, he accepted service with the Pan-American Exposition as Designing Engineer, which position he resigned to become a member of the firm of Edward J. Hingston and Olaf R. Pihl, for building the coffer-dam, foundation, piers, and abutment for a movable dam and guide cribs, and erecting Chanoine wickets, at Herr Island Lock and Dam in the Allegheny River, all under the direction of the U. S. Engineer Office at Pittsburgh, Pa.

During 1901-04, Mr. Pihl was associated with the Dravo Construction Company, as Secretary and as Engineer in charge of design and field work.

In 1905 he was taken into the firm of Cadwallader and Robinson, Contracting Engineers, of Pittsburgh, Pa. During his connection with this firm, they built the masonry arches at Lashel and Stoops Ferry for the Pittsburgh and Lake Erie Railroad, a power-house for the Pittsburgh Division of the Pennsylvania Railroad, a freight-house for the Allegheny Division of the same railroad, and double-tracked a portion of the Youngwood Branch.

On April 1st, 1906, Mr. Pihl formed a partnership with Mr. W. B. Miller, under the firm name of Pihl and Miller, Contracting Engineers, with offices in the Wabash Building, Pittsburgh, Pa. He continued with this firm until his death. During this time he completed no less than 300 contracts with railroad companies and leading steel plants in the Pittsburgh District. Mr. Pihl was particularly interested in water intake work, and designed and built some of the best plants in that section.

It has already been noted that he came from a very excellent home which left a lasting impression on him and shaped his personality. He was a gentleman in the best meaning of the word, in his tastes, and in the selection of his friends. To those only superficially acquainted with him, he appeared to be distant; to his friends, he was as true as gold and most considerate and courteous.

In addition to his ability as a constructive engineer, Mr. Pihl developed amateur photography to a very high artistic degree. Though he would never consent to make public exhibition of his efforts, his nearest friends had the pleasure of seeing much of his masterly work which was beautiful both in feeling and execution.

He did not relish service under others—in other words, he did not like to be a subordinate—and had the courage to strike out for himself as an engineer and contractor. Too few engineers possess this faculty, the lack of which keeps them from securing independence and financial success.

His chief characteristic as a contractor was to make his work of the first quality (cost not being considered); and he would not countenance or employ any but honest and capable men. He was quick to see the good in a man, and as quick to see his weakness. He was the most generous of men, and would help in any charity he considered deserving.

Mr. Pihl was energetic and progressive in constructive ideas, being well-grounded in the knowledge of theoretical engineering. He was a man of principle, and was not afraid to stand up for his convictions; whatever he undertook was accomplished in a thorough and conscientious manner. Taken as a whole, he was a true, honorable man. His friends will remember him with respect and reverence.

In August, 1904, Mr. Pihl was married, in New York City, to Sofie Christine Mellbye, of Christiania, Norway, who, with two daughters, survives him. He is also survived by his mother, six sisters, and a brother, who live in Norway.

Mr. Pihl died suddenly on October 14th, 1915, of heart failure on his way from his home to his office. He looked very strong and apparently was in excellent health. His body was cremated and his ashes were taken to Norway by his widow, to be laid at rest in the family burying ground in Christiania.

Mr. Pihl was elected a Member of the American Society of Civil Engineers on October 2d, 1889.

CHARLES HENRY PRESTON, M. Am. Soc. C. E.*

DIED APRIL 20TH, 1916.

Charles Henry Preston, the second son of Aaron Lee and Susan M. Preston, was born on Bundy Hill, Lisbon, Conn., on September 12th, 1852.

At an early age Mr. Preston showed a liking for mechanics, and began his career with the Greeneville Mills at Norwich, Conn., working as a millwright apprentice. While thus engaged he took up the study of architecture, later completing a course at the Northwestern University, in Chicago, Ill.

During his career, Mr. Preston designed and supervised the building of many of the larger private and public buildings, besides some of the most important mills, in the New England and Middle States.

Mr. Preston was not only a giant in stature, but a big man in every sense of the word. He was an optimist of the most pronounced type and, being blessed with a kind and sympathetic nature, won a host of admiring friends who will miss the hearty greeting which was familiar to all who knew him.

Besides being a member of many fraternities, he belonged to the Putnam Phalanx (a military order) and was a Thirty-second Degree Mason. He was also a member of the American Association of Architectural Engineers.

Mr. Preston had been working unusually hard, and was suffering from nervous exhaustion when he contracted the grippe; this was followed by inflammatory rheumatism, making a combination which even his strong constitution could not withstand, and he died at his home in Norwich, Conn., on April 20th, 1916, after an illness of three weeks.

He is survived by his wife and one son, Charles H. Preston, Jr., M. Am. Soc. C. E., who is a prominent civil engineer of Waterbury, Conn.

Mr. Preston was elected a Member of the American Society of Civil Engineers on October 5th, 1909.

* Memoir prepared by Charles E. Elwell, M. Am. Soc. C. E.

ISAAC RICH, M. Am. Soc. C. E.*

DIED MARCH 11TH, 1915.

Isaac Rich, son of the late Thomas and Maria L. Rich, of Brookline, Mass., was born there on October 6th, 1856. He prepared at the Boston English High School for the Massachusetts Institute of Technology, where he was a member of the Class of 1878; and the greater part of his useful and honorable life was given to the service, as a civil engineer, of various New England railroads.

His first experience came in college vacation employment as a Rodman on the surveys of the Boston, Hoosac Tunnel, and Western Railroad, between Hoosick Falls and Mechanicville, N. Y. Then, following graduation, his earlier years were divided between New England and the Far West; in 1878-79, on the Atchison, Topeka and Santa Fé, through the Royal Gorge and Grand Canyon of the Arkansas, in Colorado; in 1879-80, on the New York and New England Railroad, constructing the main line from Danbury, Conn., to Towners, N. Y.; during 1880-82, Construction and Resident Engineer on the California Southern Railroad, with supervision over four divisions; in 1882-85, Assistant Engineer for the New York and New England Railroad, with a great variety of work; during 1885-88, Assistant Superintendent, Engineer, and Accountant for the Napa Consolidated Quicksilver Mining Company at Oak Hill, Cal.

In October, 1888, Mr. Rich returned, permanently, to New England, and there was engaged in his profession until illness made his retirement imperative, in May, 1912. From 1888 to 1892, he was with the New York and New England Railroad as Assistant Engineer; from 1892 to 1895, with the Boston and Maine Railroad as Assistant Engineer and Division Engineer; and from April, 1895, until the close of his active years, with the New York, New Haven and Hartford Railroad, as Assistant Engineer on that part of the system which was the original Old Colony Railroad. In this position he performed important tasks in all the varied lines of construction and maintenance work that come to a railroad engineer, and his professional ability, personal integrity, and devotion to duty were appreciated and admired by all who knew him and his work.

Mr. Rich married Miss Alice Montague Vinal, daughter of the late Robert A. and Almira L. Vinal, in 1889, and from that time to his death resided in Somerville, Mass. Besides his widow, he is survived by a brother, Henry Rich, and a sister, Mrs. John A. E. Hussey, both of Brookline.

* Memoir prepared by J. W. Pearson, Esq., Division Engineer, New York, New Haven and Hartford Railroad Company, Boston, Mass.

Devoted to his home and to his work, content in the happiness and the occupation they afforded him, Mr. Rich had little time or inclination for other interests. Quietly genial and kind, unobtrusively helpful, always efficient, Isaac Rich will long be remembered as a good man, a loyal friend, and a capable engineer, who gave the best that was in him, always, to the work that came to his hand.

Mr. Rich became a member of the Boston Society of Civil Engineers in 1890. He was elected a Member of the American Society of Civil Engineers on May 6th, 1903.

JAMES VINCENT ROCKWELL, M. Am. Soc. C. E.*

DIED MAY 24TH, 1916.

James Vincent Rockwell, son of the late Col. James Vincent Rockwell, U. S. A., and Eckley West Rockwell, was born in Princeton, Ind., on September 22d, 1877. He spent his boyhood at the various army posts to which his father was assigned, and, like most sons of officers, could hardly be said to have had a home in the ordinary sense of the word, as applied to a definite locality. He was prepared for college at the Troy Academy, Troy, N. Y., and on his completion of the course in 1894, he entered Rensselaer Polytechnic Institute, from which he was graduated, in 1898, with the degree of Civil Engineer. In his preparatory work, as well as at Rensselaer, he stood first in his class.

Born in the service, and reared under the influence of service conditions, Mr. Rockwell found it impossible to deny the call in his blood when war with Spain broke out, in the spring of his graduating year. He enlisted as a private of volunteers on May 2d, 1898, deliberately sacrificing, as he then thought, the 3½ years of college work and his future professional career as an engineer. On July 1st, 1898, Mr. Rockwell was discharged from the service with the volunteer troops, in order to accept an appointment as Assistant Engineer in the Navy, with the rank of Ensign, and took the oath of office under this appointment on July 2d, 1898. He was permitted to return to Rensselaer Polytechnic Institute for graduation with his class, of which he was President, and was distinguished as honor man by having his diploma held until all the others had been delivered. A number of his class had enlisted for the war, and could not be present at Commencement. The Board of Trustees delivered the diplomas of the absentees to Mr. Rockwell, with the statement that they knew of none more worthy to receive them.

Mr. Rockwell served at the Navy Yard, New York, and the Navy Yard, Norfolk, Va., during the summer and fall of 1898, and was honorably discharged from the Naval Service in February, 1899. He then accepted a position as Assistant Engineer with the Chicago and Northwestern Railroad, and continued in the service of that Company until July, 1903, rising steadily in the ranks of its Engineering Staff, and enjoying the esteem of his superiors.

The first competitive examination for commission as Assistant Civil Engineer, United States Navy, under the law authorizing the grade, was held at the Navy Yard, New York, in June, 1903. Mr. Rockwell's Navy Yard service during the Spanish War had attracted

* Memoir prepared by Leonard M. Cox, M. Am. Soc. C. E.

the attention of Rear-Admiral Endicott, U. S. N., Past-President, Am. Soc. C. E., at that time Chief of the Bureau of Yards and Docks, and it was on the suggestion of that officer that he entered this examination as one of the large number of applicants. The ordeal was most severe and consumed the better part of two weeks, covering practically the entire scope of the standard technical course, as well as practical design and construction of the types of structure with which the maritime engineer has to do. Mr. Rockwell stood first in this examination and was commissioned Assistant Civil Engineer, U. S. N., with the rank of Lieutenant (Junior Grade), on June 27th, 1903. His first duty was at the Navy Yard, Boston, Mass., where he was stationed from July 27th to September 3d, 1903.

From September 8th, 1903, to October 28th, 1904, he was at the Naval Academy, Annapolis, Md., where he served as Resident Engineer on the construction of the new Academy buildings, and as Instructor in Mathematics and Physics. From October 29th, 1904, to July 24th, 1906, he was on duty at the Naval Station, San Juan, Porto Rico, where he was married in July, 1906, to Miss Isabel Romero. From August 15th, 1906, to July 25th, 1909, he was stationed at the Mare Island Navy Yard, as Senior Assistant to the Public Works Officer, Civil Engineer H. H. Rousseau, U. S. N., M. Am. Soc. C. E., and on the appointment of that officer as Chief of the Bureau of Yards and Docks, Mr. Rockwell was left in charge for one year, an unusual mark of confidence in an officer of his age and length of service.

From August 2d, 1909, to January 25th, 1910, he was on duty at the works of the General Electric Company, Schenectady, N. Y., inspecting electrical equipment for use by the Government. From February 1st, 1910, to March 23d, 1911, he was stationed at the Navy Yard, New York, as Senior Assistant to the Public Works Officer. From March 28th, 1911, to January 20th, 1913, he was on duty at the Navy Yard, Charleston, S. C., as Public Works Officer of the station. During this duty, he designed the concrete wharf and sea-wall which has recently been completed, a structure supported on pile-founded piers, built within a steel cylindrical coffer-dam, and involving difficult engineering problems. From January 25th, 1913, to April 20th, 1915, he was on duty at the Bureau of Yards and Docks, Washington, D. C., where he had charge of requisitions and certain of the contract work.

From April 30th to July 15th, 1915, Mr. Rockwell served as Senior Assistant to the Public Works Officer, Navy Yard, New York, and from July 21st, 1915, to May 24th, 1916, he was on duty at the United States Aeronautic Station, Pensacola, Fla. Upon reporting he found himself in the position of Planning Superintendent for the entire station, but despite the arduous duties involved thereby, he formed the con-

viction that all officers on duty at the Navy's only flying station should qualify as pilots, and, acting on this conviction, he immediately applied for flying work. He had completed the course of instruction and on May 24th, 1916, was undergoing one of the prescribed final tests, when the accident which caused his death occurred.

Mr. Rockwell was an able officer, a skilled engineer, and a gentleman. Somewhat reserved by nature, he nevertheless possessed qualities which endeared him to his friends. He was a man of excellent judgment and strong convictions which he never hesitated to express. He had quite a sense of humor and a quaint philosophy of life which made it impossible for him, or for those associated with him, to magnify the difficulties of any task or any problem. Perhaps his views regarding aviation duty best illustrate the man:

"Don't worry," he said in his personal correspondence, "I have passed the reckless age; I am not going into this for glory. I feel that shore-going staff officers should prepare themselves with a view to relieving line officers from shore duty in time of war—they will be needed at sea. Besides, I have an idea that my Commandant would like to see all of his officers qualify."

True to his service training, his thoughts were first of his duty, as he saw it. Civil Engineer Rockwell was not a man to take unnecessary chances—he did not want to die, but if death had to come to him, he would have wished it to be "in the line of duty." Surely those who have sacrificed themselves in developing the air branch of the service deserve a place in the memory of their fellow-countrymen with the hero who falls in battle.

He is survived by his wife and three young children; his mother, Mrs. James Rockwell, of Washington, D. C.; a brother, Charles K. Rockwell, late Captain, Corps of Engineers, U. S. A.; and a sister, Miss Helen Rockwell.

Mr. Rockwell was elected a Junior of the American Society of Civil Engineers on April 3d, 1900; an Associate Member on February 4th, 1903, and a Member on November 5th, 1907.

HENRY ROHWER, M. Am. Soc. C. E.*

DIED MAY 4TH, 1916.

Henry Rohwer, the son of Henry and Margaret Rohwer, was born on October 17th, 1847, on his father's estate near Rensburg, Holstein, Germany. This estate had belonged to the family for several centuries, the house in which Mr. Rohwer was born being some 200 years old at the time of his birth. He attended the provincial schools, and, later, Dr. Jessen's Polytechnic Institute, at Hamburg, from which he was graduated at the head of his class in 1865, receiving as a Government Prize, a position as Civil Engineer on the East Holstein Railway. On resigning this position to take up further studies at the Royal Polytechnic School, at Hanover, his Chief Engineer wrote:

"Mr. Henry Rohwer has been engaged as Civil Engineer on the East Holstein Railway, in the field, and in the office. All work entrusted to his supervision has been executed by him diligently and carefully. Mr. Rohwer is a man of unusual attainments and of a most attractive personality, an able leader and governor of men."

At the Royal Polytechnic School, Mr. Rohwer took, in addition to the regular civil engineering work, a course in Architecture, and special studies in the general principles of machinery. He stood again at the head of his class when he was graduated, with the degree of Civil Engineer, in 1869.

Broken in health from over study, Mr. Rohwer's physicians ordered a sea voyage, suggesting a trip to South America, but he decided to come to the United States. He went directly to Omaha, where he found employment with the Burlington and Missouri River Railroad in Nebraska, successively as Topographer, Engineer in charge of location and construction, Resident Engineer of that road and Acting Chief Engineer of the Omaha and Southwestern Railroad. He did his work so well that when the Chief Engineer of the Burlington and Missouri Railroad, Col. Thomas L. Doane, resigned, in 1876, to accept the position of Consulting Engineer of the Hoosac Tunnel, he recommended that Mr. Rohwer be named his successor. This recommendation was approved, and Mr. Rohwer was offered the position. He decided, however, to do private work, and opened an office in Omaha for general practice as a Civil Engineer. During this early connection with the Burlington, one of Mr. Rohwer's advanced parties was massacred, their instruments buried, and their property appropriated by the Sioux Indians. He himself was twice taken by the Indians, but escaped injury through the good offices of the interpreters.

* Memoir prepared by W. S. Dawley and John Lyle Harrington, Members, Am. Soc. C. E.

In 1877, engineering work being slack, Mr. Rohwer was one of fourteen candidates for the position of City Engineer of Omaha. He was especially recommended and vouched for by such men as Messrs. Robert R. Livingston, Surveyor-General, William Cleburn, Division Engineer, E. Lane, Engineer and Superintendent of the Building and Bridges Department of the Union Pacific Railroad Company, and the Hon. A. S. Paddock, United States Senator. He was selected and appointed by Mayor-elect, Col. R. H. Wilbur, and, two years later, was re-appointed by Mayor Chase.

While City Engineer, Mr. Rohwer resurveyed and monumented much of the city; established street grades; made a topographical map of the city; planned a sewer system and began its construction; designed water-works; in short, did everything pertaining to the Engineering Department of a young and growing community.

In 1881, Mr. Rohwer again took up railroad work, and was given charge of the location and construction of the first 50 miles of the Oregon Short Line, from Granger, Wyo., west. Before the completion of this 50 miles, he was assigned to the Portneuf River Canyon work and the construction of the line across the Bannock and Shoshone Indian Reservations. As Resident Engineer, he located the line across Snake River at "American Falls", made the triangulation, and took the necessary levels, overcoming unusual difficulties.

In 1883 he was selected by the Chief Engineer, the late Jacob Blickensderfer, M. Am. Soc. C. E., to act as Resident Engineer of the Hodges Pass Tunnel. This tunnel pierces the Uintah Range of the Wasatch Mountains, is 1 532 ft. in length, and was very difficult to construct on account of the many kinds of material and large volume of water encountered, eight separate veins of coal, with various clays, sands, and rock, being found. The tunnel was lined throughout with California redwood, and was extended on each end by snowsheds of the same material to a total length of ½ mile. The plans were well made, the work was well and carefully done, no lives were lost, and many compliments were paid to the engineer and contractor. On the completion of this tunnel in 1884, Mr. Rohwer, against the wish of his Chief Engineer, decided to resign and again take up private practice. He returned to Omaha where he became interested in the making of brick, designing and patenting a "continuous brick kiln" which has special merit and gives much satisfaction where fuel is expensive. He constructed a number of these kilns throughout the United States.

In 1885, Mr. Rohwer again returned to railroad work and, under Mr. S. H. H. Clark, President of the Missouri Pacific Railroad, located and built the Omaha Belt Railway and the Missouri Pacific Extension to Lincoln, Nebr. In 1887 he was made Engineer in Charge of Maintenance of the Missouri Pacific Railway and its branches, which

position he held until March 1st, 1901, when he was advanced to the position of Chief Engineer of the Missouri Pacific Railway and the St. Louis, Iron Mountain and Southern Railway, Leased and Operated Lines. The following is a part of the work done while Mr. Rohwer was Chief Engineer of these lines: Plans and estimates for revision of alignment and reduction of gradients on 900 miles of road, this work being completed as planned on some 550 miles; surveys for some 1 100 miles of new line, of which about 700 miles were built. These new lines include: The Memphis, Helena and Louisiana Railway, 230 miles in length, with its two large draw-bridges (440 ft. and 370 ft.) across the Arkansas and White Rivers; the Eldorado-Bastrop Railway, 44 miles in length, with the Ouchita Draw and its 2 miles of trestle approach; and the White River Railway, 240 miles in length, through the Ozarks of Missouri and Arkansas, costing more than $8 000 000. This line, for most of its length, was of heavy construction, necessitating five tunnels, totaling 2 miles in length, and many steel bridges and viaducts, the latter often reaching a height of 100 ft. or more. This is a road of great scenic beauty.

Mr. Rohwer represented his Company on the Board of Engineers for the Thebes Bridge, and acted as Chairman of that Board. He was at the same time a member of the Board of Chief Engineers for the Kansas City New Union Depot and Terminals.

After 1906 he maintained a consulting engineer's office in St. Louis, devoting his time to consulting and expert service, making reports on railroads, water-works, power-plants, etc. He acted as Consulting Engineer for investors, financial institutions, and municipalities. In the latter capacity he reported on the location and cost of the municipal "Free Bridge," and his report, published by the City, decided the location of that bridge.

Mr. Rohwer was an early advocate of good roads. His address at the first Good Roads Convention of the State of Missouri, held at Sedalia, in January, 1893, was published in full in its proceedings, and contained the strongest arguments advanced. He was at that time called by his friends "The Father of Good Roads."

Mr. Rohwer was a Charter Member of the American Railway Engineering Association; a Member of the Engineers' Club of St. Louis; a Member of the Society of German Engineers, of Berlin, Germany; and a Charter Member of the Fraternity "Hannovera", founded in 1866, of Hanover, Germany. He served as a member of the Roadway Committee of the American Railway Engineering Association, and was active in the St. Louis Club.

He was always a hard worker, and his associates recognized his superior ability for carrying out those undertakings which required

continuous and careful attention as well as unusual technical attainments. One contractor writes:

"As a contractor, I had the good fortune to work under Mr. Henry Rohwer during the greater part of the time that he was Chief Engineer of the Missouri Pacific and Iron Mountain Railroads. He was an honorable man, slow to anger, scrupulously just, and always ready to commend work well done. The majority of contractors were always glad to do work under him, knowing they would receive fair treatment at his hands, consequently, construction work under his management was prosecuted promptly and to the satisfaction of all parties concerned."

Aside from his eminence as an engineer, Mr. Rohwer was a good man and greatly devoted to his home and family. He was married on October 8th, 1873, to Anna Sievers, of Omaha, Nebr., who survives him, with three of their five children, Oscar H. D., Henry G. A., and Anna Louise.

His only recreation was taken with his family. He kept them near him whenever possible. He had them for a year at his camp when constructing the Hodges Pass Tunnel. At that time there were no white settlers in Southwestern Wyoming, and the Indians of the reservations crossed were not at all times friendly. While an officer of the Missouri Pacific Railway, it was his custom to take one or more of his family with him when making trips down the line.

At the funeral services, his pastor spoke of him as the most modest of men. His daughter writes of him as follows:

"The devotion of Mr. Rohwer to his family was exemplary, and his life was one beautiful harmony. In the gentleness so characteristic of himself, despite weakness, pain, and growing apprehension, the only greeting ever received was a smile. His death has its own peculiar pathos, and he met it, as he met all else in life, with a resolute determination. There was no outward expression of inward lassitude, nor was there a portrayal of weakness or of waning fortitude. His was a long heroic uncomplaining struggle, and every obligation on the part of his family was lovingly met. Sclerosis of the arteries, affecting the heart, caused undescribable suffering, both mental and physical. He bade the members of his family farewell the day before his death, which occurred on the afternoon of May 4th, 1916. The remembrance of his personal qualities and the beauty and rarity of his spirit, together with the love and esteem in which he was held by all who knew him, to some degree, temper the grief of those who mourn him."

Mr. Rohwer was of a quiet, unobtrusive disposition, and did not seek the world's plaudits; but he fought a good fight, lived an honorable and useful life, and earned the world's esteem.

Mr. Rohwer was elected a Member of the American Society of Civil Engineers on April 1st, 1903.

FRANCIS WINTHROP SCARBOROUGH, M. Am. Soc. C. E.*

DIED DECEMBER 24TH, 1915.

Francis Winthrop Scarborough was born in Cincinnati, Ohio, on September 6th, 1865. He was graduated from the Rensselaer Polytechnic Institute in 1888, and immediately entered the service of the Chesapeake and Ohio Railway Company as an Assistant Engineer. In July, 1891, he was appointed Division Engineer in charge of maintenance on various Divisions and remained in this position until April, 1899, when he was made Engineer of Bridges and Signals. In this capacity, he had full charge of the design and construction of all bridges and signal plants for the Company.

In January, 1902, he was appointed Chief Engineer of Maintenance of Way and Structures, with jurisdiction over all the lines operated by the Company, which position he held until he resigned in 1908. During this period, Mr. Scarborough had charge of the construction of the Richmond Viaduct and Terminal, one of the greatest railroad improvements executed in that section of the country, the general plan of which was initiated by Harry Frazier, M. Am. Soc. C. E.

After his resignation in 1908, Mr. Scarborough became actively engaged in the operation of coal mines in the New River region of the Chesapeake and Ohio Railway, with headquarters at McDonald, W. Va. Later, he returned to Richmond, Va., where he practiced his profession as a Consulting Engineer. At the time of his death, he was President of the Virginia Bonded Warehouse Corporation, and was also engaged in the development of plans for water and sewerage systems for Charlottesville, Va.

As an officer of the Chesapeake and Ohio Railway Company, Mr. Scarborough commanded the confidence of the executives, and conducted his organization with marked ability. Added to his scientific attainments, which were notable, and to his fine sense of organization, which was of a high order, was the unique personal gift of eliciting the complete loyalty, and even the affection, of his subordinates. Tireless himself as a worker, he imbued others with a like spirit. Thus, he became a power on that road.

Mr. Scarborough was peculiarly fearless in his mental characteristics. He was always reasonable and open to conviction, yet, when he saw what seemed to him to be the truth, he maintained his position with the utmost firmness and with unfailing courtesy. He was thoroughly fitted for the high calling of his choice; fitted by Nature for any high calling.

The record of his life is brief, but to those who knew him and loved him, that record would be incomplete—even barren of the truth—

* Memoir prepared by Messrs. C. J. Roelker and James P. Nelson, Richmond, Va.

did it not contain some expression of the love that went with him wherever he met with his fellow men.

His opinions were sought, and were the guides for men handling important matters. All of this is stated of the accomplished engineer, the able leader and governor of men, the counsellor in his Profession; but those who knew him as the man think of him as the large-hearted, generous, loyal, beloved friend; as the unassuming, open-handed, delightful companion, ever speaking of all men only that which was kind, with a soul devoid of malice toward any man.

The last months of Mr. Scarborough's life were clouded by sickness, but even then his spirit never lost its brightness, his courage never failed. As men should, he met his last earthly summons, and when the message came, made answer without fear. He died in New York City on December 24th, 1915.

Mr. Scarborough was a member of the Westmoreland and Commonwealth Clubs, and of the Country Club of Virginia. He served two years in the Commonwealth Club as Vice-President. He was also a member of the American Society of Mechanical Engineers, the American Institute of Mining Engineers, the American Society for Testing Materials, the American Railway Engineering Association, and the Delta Phi Fraternity.

Mr. Scarborough was elected a Junior of the American Society of Civil Engineers on September 3d, 1890; an Associate Member on March 6th, 1895; and a Member on February 2d, 1904.

JOEL HERBERT SHEDD, M. Am. Soc. C. E.*

DIED NOVEMBER 27TH, 1915.

By the death of J. Herbert Shedd, the Engineering Profession loses a man of distinction. Early in life, Mr. Shedd established a reputation as an engineer of superior ability in hydraulic and sanitary work. He was born on May 31st, 1834, and began his professional life in 1850, when he entered, as a student for three years, the engineering office of Messrs. T. and J. Doane, at Charlestown, Mass. Before the three years were completed he was given charge of important railroad work in Indiana. In 1856 he opened an office in Boston for the general practice of his profession, but specialized in hydraulics and sanitary engineering.

Mr. Shedd retained his office in Boston for forty years. Many engineers, in practice to-day, scattered over our country, received part of their training under him. When he decided to give his attention to the designing and construction of the Providence Water-Works, he left the Boston office in charge of his partner, Mr. Edward Sawyer, at which time sixteen young men were studying engineering there. This was not long before the Institute of Technology was established.

In selecting hydraulics for his special branch of professional work, Mr. Shedd made no mistake. His aptitude and ability in this line were soon recognized by public officials. As early as 1860 he was appointed by Gov. Andrew, of Massachusetts, as Commissioner on the Concord and Sudbury Rivers. In June, 1876, on the establishment of a Board of Harbor Commissioners for the State of Rhode Island, Mr. Shedd was appointed a member, was elected Chairman, and held that position continuously until the end of his life. He was Commissioner from the State of Rhode Island to the Paris Exposition in 1878. Later, he was engaged in other important service for the State: as Chairman of the Rhode Island Section of the Interstate Commission to establish the boundary line between the States of Rhode Island and Connecticut, about 1885; as Chairman of the Rhode Island Section of the Interstate Commission to establish encroachment lines in the Pawcatuck River, about 1887; as a member of a Commission to inquire into the "Desirability, Necessity, and Cost of Either Making Changes in the Present Stone Bridge across Sakonnet River, or of Erecting a New Structure", 1902 to 1904, and as a member of a Commission for a new bridge, 1904 to 1910.

Among all the works devised and built by Mr. Shedd, those at Providence are best known. In the opening statement of his report to the "Committee of the City Council on a Supply of Pure Water

* Memoir prepared by O. Perry Sarle, M. Am. Soc. C. E.

for the City of Providence" of October, 1868, he has recorded his engagement in the following words:

"In the month of October, 1866, I was called upon by you to make a professional examination of all possible sources of supply, with a distinct understanding that this examination should be entirely independent of all previous investigations and reports. To this end I was kept in ignorance of what had been done in the past, and you have carefully refrained from giving me your impressions, if any you have, for or against any particular plan. I have thus been left free to consider the various possible plans in purely scientific light, without personal prejudice from any source."

This Committee, in its statement transmitting his report to the City Council, says:

"Mr. J. Herbert Shedd of Boston, an engineer of extensive reputation and business in Massachusetts, has served us faithfully. He has not only brought scientific attainments to bear upon the great question, but has made himself so familiar with our city and its needs in this regard as to become to all intents a citizen, except in prejudices."

In 1874, when the Providence Water-Works were nearing completion, on February 24th, the *Providence Journal* says, in its editorial columns:

"As the official term of the Board of Water Commissioners expires this week, the present has seemed to us a fitting time to review their work and sum up the result of their labors, * * *." "Desiring that the examination of our water-works and the resulting report for the *Journal* should not only be thorough, intelligent and comprehensive, but also be absolutely unbiased and impartial, we requested aid of the Hon. William Aspinwall, of Brookline, Mass., a member of the Water Board of that town, and a gentleman admirably qualified by cultivation, experience and observation in this and other countries, to pass judgment upon works of this description, and whose high character debars any suspicion of partiality or prejudice."

Mr. Aspinwall, in his report, printed in the *Journal* of the same date as the foregoing editorial, refers to the Chief Engineer in the following language:

"The Commissioners, with all their intelligence, faithfulness, and diligence, could never have reached the results attained had they not been so fortunate in their selection of a Chief Engineer.

"When Mr. Shedd first came to the work of the city, in 1866, to make the preliminary examinations reported in 1868, he brought with him, although still quite a young man, a most enviable reputation for a rare combination of the qualities essential to the attainment of the first rank in his profession. He already stood among the first engineers in Massachusetts for theoretical and practical knowledge of his business, as well as for an enthusiastic love for it. He was, besides, distinguished for his great executive ability and powers of systematic arrangement, and lastly for his perfect truthfulness and incorruptible

integrity. This well earned reputation, I am happy to learn, has been fully sustained in his dealings with the people of Providence, and is cordially admitted, even by those who differ from him in opinion. The Chief Engineer has been the inventing and suggesting brain, as well as the strong right arm which has aided the Commissioners in deciding and then in executing the plans, first for the water-works and then for the sewers of Providence. Everything has passed under his supervision, and much if not most of the system which is so conspicuous in all the executive departments is due in great measure to him."

While Chief Engineer of the water-works Mr. Shedd designed and partly built the sewerage system in Providence. On retirement from this work he entered upon private practice in Providence, but returned to the city as City Engineer in 1890, which position he held for seven years. He also remained with the city for the two years next following this engagement, as Consulting Engineer, when the sewerage works were substantially completed.

His private practice succeeding his service with the City of Providence brought to him numerous water-works and sewerage plants to be investigated and developed. His knowledge and practice in the field of stream flow, and his general experience in hydraulics, led to work with reference to damages caused by the diversion of water for public supplies, which, from about this time forward, constituted a very important part of his professional practice. Some of the notable cases for which he did much important and successful expert work were those against the City of Pawtucket, R. I., for the diversion of the waters of Abbotts Run; cases against the City of Worcester, Mass., for the diversions of Tatnuck and Kettle Brooks; cases against the City of Woonsocket, R. I., for the diversion of the waters of Crook Falls Brook; cases against the Warwick and Coventry Water Company, for the taking of Carrs Pond; cases involved by the establishment of the Wachusett Reservoir, in Massachusetts, and numerous others in this same class involving damages of a similar nature.

His knowledge of stream flow caused a demand on his skill for the purpose of fixing responsibility for failure of works to withstand safely the discharge of streams in flood time. Among his commissions in this line is notably the breaking of the dam of the Diamond Hill Reservoir, in the great freshet of February, 1886, which caused extensive damage in the valley of Abbotts Run.

His practice in water-power work involved the examination of streams and catchment areas, the estimation of the power of streams, and the design and development of water-power plants. He made the complete study and development at Rumford Falls, Me., which power was entirely unimproved when he began work thereon.

Mr. Shedd was commissioned by the Norwich Water Power Company, at Norwich, Conn., to determine the quantities of water used

by the several mills on the canal, and to establish a system for measuring the water. This commission was carried out, and the system established has been in use since about 1888. The charges for water used in the several mills are based on the measurements obtained by the use of this system. He also made similar investigations for the Union Water Power Company, at Lewiston, Me., and for the canal at Windsor Locks, Conn.

Although Mr. Shedd was not a prolific writer, he contributed much valuable literature on engineering subjects in the form of reports, some of which have been regarded as classics. He wrote the section on Rain and Drainage in French's "Farm Drainage", 1859, and the report on the Ventilation of Representative Hall of the State House in Boston, 1865. In 1880 he was appointed a member of a Committee of the American Society of Civil Engineers with reference to fixing a "Uniform System for Tests of Cements". He addressed the American Paper and Pulp Association in February, 1889, at its annual meeting in New York, on the subject of "Application of the Principles of Water Storage in New England Rivers". In recognition of his literary and scientific attainments, Brown University, in 1894, conferred upon him the honorary degree of A. M. For many years he was a member of the Visiting Committee to the Mechanical Engineering Department of Brown University.

Mr. Shedd always took much interest in the solution of problems and the design of devices for meeting cases which were out of the line of the ordinary development of work. He did not, however, make many attempts to secure his inventions by patents. He devised the Shedd water meter, which he patented. This proved to be a most excellent device, but was not developed commercially owing to the greater attraction of his professional life with which he was fully occupied. He was one of the patentees of an automatic flash-board, which was put in service at the lower dam on the Quinebaug River, near Taffts, Conn., about sixteen years ago, where it has given excellent service.

The last fifteen years of his life were largely devoted to consulting and expert service. His broad experience in his chosen field developed a demand for his opinions relating to valuations of water-works and water-power properties, and the value of water, to an extent rarely found in any one man's career. As an expert witness in contested cases, he has rarely been equalled.

In going over this brief sketch of a very busy life it is to be noted that very early, almost at the beginning, he had inspired in those with whom he came in contact a firm belief in his honor and integrity, and that all through his professional career of nearly sixty-six years these features of his character were constantly maintained, this maintenance

being shown by the importance and magnitude of the works and commissions with which he was entrusted. There is an echo of the same high regard in a recent letter, in which it is said:

"We both had a great personal liking for Mr. Shedd, and were on most friendly and cordial terms with him, and the more we knew him the more we respected and admired him. He was open and frank, and was incapable of deception. We thought him one of the best witnesses we ever put on the stand in the line of his profession."

Another has said:

"From Maine to California, during the half century and over that he was prominently in active practice, Mr. Shedd's services were in demand, and during all that time his standing remained high and his reputation unsullied.

"J. Herbert Shedd is dead, but the many public works that were designed by him and constructed according to his directions, will, as imperishable monuments, live after him for centuries to come."

Those who knew Mr. Shedd intimately take just pride in his excellent work and in the fact that it was accomplished with so noble a record from the beginning to the end.

Mr. Shedd was elected a Member of the American Society of Civil Engineers on September 15th, 1869.

EDMUND JOB STEERE, M. Am. Soc. C. E.*

DIED JULY 18TH, 1914.

Edmund Job Steere was born in Providence, R. I., on June 13th, 1874. He studied Civil Engineering at Brown University, from which he was graduated in 1894 with the degree of Ph.B.

During the summer vacations of 1891, 1892, and 1893, Mr. Steere was employed in the office of the City Engineer of Providence, mostly on highway work. After his graduation in 1894, he was appointed First Assistant to the Resident Engineer on the construction of the Red Bridge over the Seekonk River, in Providence, and, in 1895, he served as Resident Engineer during the construction of the Exchange Bridge over the Providence River. From 1896 to 1898, he was engaged, as Resident Engineer, on the construction of stone arch bridges in Roger Williams Park and in Davis Park, in Providence. In 1898, he was employed as Engineer on the building of retaining walls along the Providence River and also on the Weybosset Bridge over that river. In 1899, he was appointed City Inspector of underground electric and telephone conduits for the City of Providence, which position he retained until 1901.

In 1901, Mr. Steere was employed as Civil Engineer by the J. W. Bishop Company, General Contractors, on the Lonsdale Bleachery. In 1902, he was made Construction Engineer and put in charge of the construction of the power-house and 1 800-ft. dam at Danville, Va. In 1904, he was made Superintendent of Construction and had charge, for his Company, of the construction of mills, power-houses, dams, and bridges, in various parts of the United States.

From 1906 to 1909, Mr. Steere was engaged as Superintendent of Construction and Contractors' Engineer on the construction of Post Headquarters Building, and several small buildings, at the United States Military Academy, West Point, N. Y. He had full charge of this work which was valued at $500 000.

After the completion of his work at West Point, Mr. Steere was offered a position on the Panama Canal and had offers from contracting firms of national repute from all over the United States. He refused them, however, to become, in 1909, Chief Engineer and General Superintendent of the J. W. Bishop Company. In this position he had charge of the design and construction of mills in Rhode Island, Massachusetts, and Connecticut; sewerage systems for New Bedford, Mass., and Providence, R. I.; a pipe line in Warren, R. I.; power-houses at Danielson, Mechanicsville, Ashton, Milton, etc.; and business blocks

* Memoir prepared by the Secretary from information furnished by Henry W. Ballou, Assoc. M. Am. Soc. C. E., and on file at the Society House.

in Providence and New Bedford. He had charge of from thirty to forty construction jobs a year, averaging in value from $1 000 000 to $2 000 000.

In 1914, Mr. Steere was appointed the first Commissioner of Public Buildings under the City Government of Providence, R. I., which position he held at the time of his death which occurred from drowning, on July 18th, 1914, in an attempt to save the life of a friend.

Mr. Steere was a man of few words, one whose instinct was to listen, to think, to act, and to give orders quietly. Of great physical strength, he was deliberate and almost slow in his bodily movements, except in moments of emergency when he revealed a body and muscles worthy of a tiger. An idea of his capacity for work may be had from the fact that he had directed millions of dollars worth of construction before he was forty years old, and often drove his automobile from 150 to 200 miles per day, superintending the work of 1 200 men on from six to ten jobs scattered over Rhode Island and adjoining parts of Massachusetts and Connecticut.

Mr. Steere was a typical Yankee in that he always answered a question by asking another, and his statements were made preferably in an interrogative form. He was ill at ease if in any way conspicuous, and although he had many acquaintances, his intimate friends were few. He was devoted to his family and denied himself to his friends in order to spend the time with his two sons who survive him.

Mr. Steere was elected a Member of the American Society of Civil Engineers on October 1st, 1913.

ERNEST FREDERICK TABOR, M. Am. Soc. C. E.*

DIED AUGUST 20TH, 1916.

Ernest Frederick Tabor was born at Worcester, Mass., on February 26th, 1866. In his youth, his parents moved to California, where he received his education in the public schools and at the University of California.

At the age of 21, Mr. Tabor began his engineering career, in the office of the San Diego Flume Company, and was connected in various engineering capacities with a number of large enterprises in Southern California. In 1890 he made the preliminary investigations for the Escondido Irrigation District, and in 1894-95, as Chief Engineer, he had charge of the construction of the Escondido Dam, a rock-fill structure of note, which has been described in many engineering papers. He also had charge of the construction of canals and pipe lines to reach about 3 000 acres of settlers' lands.

During 1900 Mr. Tabor was engaged in the location and construction of a 20-mile, wood-stave pipe from the Lower Otay Reservoir to San Diego. This work included several tunnels and an earth dam. Mr. Tabor's general engineering practice in Escondido, Cal., included work on street grades and water-works for the Towns of Escondido, Elsinore, and San Jacinto, Cal. In 1902, he had charge of the operation of the works of the Escondido Irrigation District, but resigned later to take up similar work with the Sweetwater Water Company, at National City, Cal.

In 1904, Mr. Tabor resigned his position with the Sweetwater Water Company to enter the service of the United States as Assistant Engineer in the Reclamation Service, and after 6 months he was appointed Engineer. The next three years were spent on assignments on the Shoshone Project, Wyoming, where his principal work was the location and construction of the Garland Main Canal and some of its principal laterals. He was then placed in charge, as Project Manager, of an irrigation project on the Flathead Indian Reservation, with headquarters at St. Ignatius, Mont., which position he held until his death. The work there was of a pioneer character, and included topographical surveys and studies of stream flow, followed by surveys and formulation of plans for serving about 150 000 acres of irrigable land, and using the flow of about sixty small streams for this purpose, making a very complicated system. Mr. Tabor considered the development of these plans the most difficult and important work he had ever undertaken. While in charge of the Flathead work, he was also occasionally called on for consultation regarding the principal

* Memoir prepared by A. P. Davis, M. Am. Soc. C. E.

canals on the Blackfeet, Milk River, Shoshone, and Sun River Projects, of the Reclamation Service, as he was considered an expert specialist in canal location and construction.

Mr. Tabor died at St. Ignatius, Mont., from peritonitis.

On May 10th, 1893, he was married to Miss Gertrude M. Smith, who, with two children, survives him.

Mr. Tabor was elected a Member of the American Society of Civil Engineers on May 1st, 1907.

MORTON LOUDON TOWER, M. Am. Soc. C. E.*

DIED APRIL 17TH, 1915.

Morton Loudon Tower was born at Boston, Mass., on January 5th, 1870. He was the only son of Major Morton Tower and Annie Mason Loudon Tower, who, in 1874, moved to the Pacific Coast and settled at Empire, Ore. Through his father, he was a direct lineal descendant of John Alden and Priscilla Mullins who landed from the *Mayflower* at Plymouth in 1620. His father was a Major of Volunteers in the Civil War, and, subsequent to his removal to the Pacific Coast, was for many years Collector of Customs at Coos Bay, Ore. His maternal grandfather, Thomas Loudon, of Glasgow, Scotland, and, later, of Liverpool, England, was a mechanical engineer, inventor, and iron-maker, and helped construct and place the engines in the first steamer built by the Cunard Line to cross the Atlantic.

Mr. Tower attended the public schools of Oregon. With the exception of a few months of special instruction at the University of California in 1888, his well-balanced technical knowledge was acquired in the school of experience and by extensive reading. In his sixteenth year, he began to acquire engineering experience on surveys. From March, 1889, his employment on engineering works was practically continuous; and from March, 1891, until his death in April, 1915—24 years—his service was with the Corps of Engineers, United States Army.

Prior to his entry into the Government service, he was, from November, 1886, to August, 1887, Rodman and Chainman with Mr. N. P. Granger, of San Fernando, Cal.; March to November, 1889, General Field Assistant with Lucas Brothers, Bridge Engineers; from January to June, 1890, Chainman and Rodman, Coos Bay, Roseburg, and Eastern Railway; and from June, 1890, to March, 1891, Transitman, Topographer, Sextant Observer, Leveler, etc., with Mr. A. J. McMillan.

In the Government service, the works to which Mr. Tower was assigned were varied. From March, 1891, to November, 1901, as Junior Engineer, most of the time in local charge of works, he was engaged on river surveys and on dredging and the construction of jetties. He designed and constructed a combined dredge and snagboat and operated it on the Coquille River and in Coos and Tillamook Bays, Oregon. From November, 1901, to July, 1905, he was engaged on hydrographic surveys of, and projects for improving, several Pacific Coast harbors and rivers in Oregon and Washington, among which were Tillamook and Coos Bays, Columbia, Siuslaw, Umpqua, and

* Memoir prepared by H. H. Wadsworth, M. Am. Soc. C. E., and Lee S. Griswold, Assoc. M. Am. Soc. C. E., a Committee of the San Francisco Association of Members of the American Society of Civil Engineers.

Coquille Rivers, Oregon, and Lewis River, Washington. On jetty construction and fortification at the entrance to the Columbia River, he served under G. B. Hegardt, M. Am. Soc. C. E., at Fort Stevens. At the entrance to Coos Bay, Coquille River, and Siuslaw River, he was in local charge of jetty construction.

In July, 1905, at the instance of Colonel (then Captain) William W. Harts, M. Am. Soc. C. E., Corps of Engineers, U. S. Army, Mr. Tower was transferred to San Francisco for duty under the California Débris Commission. He made surveys of some of the tributaries of the Sacramento River, which were surcharged with débris (mining tailings) resulting from early extensive hydraulic mining operations, and prepared plans for restraining further movement of the débris into navigable waters. He also inspected many hydraulic mines and reported on the restraining works to be prescribed by the Commission for construction by operators prior to the issuance of licenses for mining.

In January, 1907, Colonel (then Major) C. H. McKinstry, M. Am. Soc. C. E., a member of the California Débris Commission, and also Engineer Officer in charge of lighthouse construction in the San Francisco District and of the fortifications of San Francisco Harbor, secured the services of Mr. Tower as Superintendent in the Lighthouse Department. While on that duty, he made surveys of lighthouse sites and designed foundations for light stations. From February, 1908, to August, 1910, he was in local charge of construction and of other field work at Forts Baker, Barry, McDowell, Scott, Miley, and Mason, San Francisco Harbor, except the period from August to December, 1908, when he was assigned to duty at Honolulu, Hawaii, in charge of harbor dredging.

From September, 1910, until his death, Mr. Tower had charge of the construction of the jetties at the entrance to Humboldt Bay, California. Humboldt Bay, on which is situated the City of Eureka, is the most important harbor between San Francisco Bay and the mouth of the Columbia River. The maintenance of safe depths across Humboldt Bar has been difficult and expensive, frequent severe storms having repeatedly beaten down the jetty enrockment and carried away the trestle used for its construction. Mr. Tower conceived the idea of concreting the top surface of the rock jetty as fast as it was raised to grade, and of anchoring the construction track in the concrete, using specially designed traveling derricks to handle the construction materials. This method of construction has proved highly successful.

On Thursday, April 15th, 1915, while proceeding with Lieut.-Col. Thomas H. Rees, Corps of Engineers, U. S. Army, and two other men, to inspect the quarry from which rock was secured for the jetty, the motor track car on which they were traveling jumped the track on a trestle and precipitated the passengers some 15 or 20 ft. down

into a gulch among boulders. The accident was witnessed by some of the quarrymen, and the injured men were speedily conveyed to a hospital at Eureka. Two of the men escaped with comparatively slight injuries, Col. Rees suffered a broken leg, but Mr. Tower had three ribs broken and was otherwise internally injured, and died two days later. As emblems of mourning, and of the esteem in which he was held at Eureka, the flags of the Federal and other public buildings, and on the bay shipping generally, were placed at half mast. He was buried at Marshfield, Ore.

Mr. Tower was married at San Francisco, Cal., on November 16th, 1894, to Cornelia Maud Armes Smith, who, with three daughters, Gladys Fredricka, Ella Gwynedde, and Cornelia Morton, survives him.

He was a man of most genial disposition, delighting greatly in associating with his friends, of whom he had many, and with his fellow engineers. He contributed a paper on bar harbors on the coast of Oregon,* as well as a discussion on a paper on concrete piles,† to the *Transactions* of the Society.

Mr. Tower was elected an Associate Member of the American Society of Civil Engineers, on May 6th, 1908, and a Member on July 1st, 1909. He was also a member of the Benevolent Protective Order of Elks, Eureka Lodge, No. 652.

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* "Notes on the Bar Harbors at the Entrances to Coos Bay and Umpqua and Siuslaw Rivers, Oregon", *Transactions*, Am. Soc. C. E., Vol. LXXI, p. 349.

† "Concrete Piles", by Howard J. Cole, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXV, p. 488.

STEVENSON TOWLE, M. Am. Soc. C. E.*

DIED FEBRUARY 14TH, 1916.

Stevenson Towle, the son of Jeremiah and Jane (Abeel) Towle, was born in New York City, on July 29th, 1837. In 1822, Jeremiah Towle had removed from New Hampshire to New York City where he became prominent in municipal affairs. He selected and planned the present water supply, served as Commissioner of Charities and Schools for many years, and was mainly instrumental in the establishment of the present park and ward school systems of the City.

Stevenson Towle was educated in the public schools of New York City, having been among the pupils first registered in the First Ward School. He continued his studies at the Free Academy, now the College of the City of New York, from which he was graduated.

Having studied Civil Engineering, Mr. Towle was made City Surveyor of New York in 1857, and, in 1860, the Mayor appointed him to visit and examine the sewerage systems of European cities for the purpose of obtaining the information on which the present sewer laws of the city are based.

From 1870 to 1886, Mr. Towle was Chief Engineer of the Department of Sewers of New York City. In addition to this work he also, in 1871, served, with Generals McClellan and Franklin, on a Commission appointed to lay out Long Island City; and, in 1883, he planned and built the first cable railroad (Tenth Avenue) in New York City.

In 1886, Mr. Towle became Consulting Engineer for the Broadway Arcade Rapid Transit Railroad, and, in 1887, he was appointed by Mayor Hewitt as a member of the first Rapid Transit Commission. In the same year he visited Europe, by appointment of Mayor Hewitt, and investigated the question of improved pavements throughout the various cities.

In 1888, Mr. Towle was made Park Commissioner of New York City, and, in 1889, was appointed Consulting Engineer for the Department of Public Works, in special charge of improved street pavements. He held this position until April, 1897, during which time he introduced a system which is now extensively used throughout the United States.

In April, 1897, he was appointed Consulting Engineer of the Sewer Department in connection with the construction of the Rapid Transit Tunnel. In 1902, Mr. Towle had a stroke of apoplexy, which left him subject to heart disease, and it was an attack of this trouble that

* Memoir prepared by the Secretary from information furnished by Charles S. Towle, Esq., New York City.

caused his sudden death at his home, the Brevoort Farm, at Mamaroneck, N. Y., on February 14th, 1916.

Mr. Towle was an extensive traveler, and had made investigations on subjects pertaining to his Profession on all the Continents. He was engaged on many of New York's transportation and sanitation projects. He was a Director of the Institute of the Deaf and Dumb and a Member of the Scotch-Irish Society.

He was married on October 12th, 1863, in New York City, to Mary Stewart, daughter of Henry and Jane (Stewart) Brevoort, who was a member of one of the oldest Knickerbocker families of that city. He is survived by three sons and five daughters.

Mr. Towle was elected a Member of the American Society of Civil Engineers on February 19th, 1868, and served the Society as Director during 1887 and 1888.

GEORGE WASHINGTON VAUGHN, M. Am. Soc. C. E.*

DIED FEBRUARY 3D, 1916.

George Washington Vaughn was born at Perry, N. Y., on November 24th, 1829.

He began his engineering career about 1854, when he made the plans and surveys for bringing from the mountains of California what was perhaps the first water supply for the purpose of hydraulic mining. This work was followed by plans for a series of dams in various California streams. He then took up Government land surveying in the Northwest.

In the summer of 1857, Mr. Vaughn was engaged as Levelman on the location of the Winona and St. Peter Railroad, the first railroad in Minnesota, and served as Division Engineer on the construction of that road until April, 1858.

On the completion of this work, he accepted a very lucrative offer to take charge of the construction of levees on the Mississippi River in Arkansas. While he was engaged on this work, the Civil War broke out, and he enlisted in the Federal Army and was assigned the management of the collection of rentals of business and public buildings in Memphis, turning the collections over to the War Department.

After the war, Major Vaughn, with a few others, decided to locate a town in the State of Kansas, and accordingly he surveyed and laid out the Town of Ellsworth, named for Col. Ellsworth, one of the first to fall in the Civil War. It is now the seat of Ellsworth County. The first site chosen for the town, being too near the river, was threatened with floods. The town was moved, therefore, and Major Vaughn's engineering ability was called into play to devise means of moving the houses back to the new site, which he had already surveyed and which is the location of the present city.

In the early days of Ellsworth, Major Vaughn and others were compelled to become vigilantes in order to rid the community of gamblers, murderers, and cut-throats by the primitive method of hanging, and after order was restored, respectable people were encouraged to settle there.

Major Vaughn decided, however, that Leavenworth was more desirable as a residence, on account of the better educational advantages open to his children, and in 1869, he purchased a home in that city, which he continued to occupy until his death.

He then took up railroad engineering, and from April to August, 1869, served as Division Engineer on the Leavenworth, Atchison and Northwestern Railway, now part of the Missouri Pacific System. In

* Memoir prepared by the Secretary from information furnished by J. M. Willard, Esq., and on file at the Society House.

1870 and 1871, he was engaged as Chief Engineer of the Kansas Central Railway, a narrow-gauge road running west from Leavenworth. On the completion of this road, Major Vaughn served until 1874 as Chief Engineer of the Wyandotte, Kansas City and Northwestern Railway (now also a part of the Missouri Pacific System) running from Kansas City to Lexington, Mo. In 1875 and 1876 he was Superintendent of the same line, and in 1878 he was again appointed Chief Engineer of the Kansas Central Railroad.

In 1879, he was engaged as Resident Engineer on the Denver and Rio Grande Railroad, in charge of the location and construction of two lines over the Continental Divide, one at Ten-Mile Pass and the other at Marshall Pass.

In 1880, Major Vaughn went to Mexico as Chief Engineer of the Northern Division of the Mexican Central Railway and of the Mexican National Railway north of the City of Mexico where he made his headquarters, completing the line to San Miguel and a branch to Morelia.

In 1886 and to April, 1887, he was Chief Engineer of the Leavenworth, Northern and Southern Railway, now a part of the Santa Fé System, and from April, 1887, to December, 1890, he was Assistant Chief Engineer of the Chicago, Santa Fé and California Railway, a line connecting Kansas City with Chicago, Ill.

On the completion of this line Major Vaughn was made Consulting Engineer of the Atchison, Topeka and Santa Fé Railroad. He served as Vice-President and Chief Engineer of the Santa Fé, Prescott and Phœnix Railway, with headquarters at Prescott, Ariz., from 1892 to 1895, but he did not like the routine of the office and resigned to return to his home in Leavenworth, Kans.

In 1897, he was called to Chicago to take charge of the work known as the 16th and Clark Streets Track Elevation. This comprised the elevation of the tracks of nine railroads operating 1 000 trains per day, and was accomplished without the stoppage of traffic during construction. It was considered one of the greatest engineering achievements of its kind in the world.

On the completion of this work in 1899, Major Vaughn was appointed Engineer in Charge of the Joint Track Elevation for the Illinois Central, the Atchison, Topeka and Santa Fé, and the Chicago and Alton roads, to extend the track elevation from 18th Street west 3 miles, at a cost of $1 000 000 per mile. This work was completed in 1905. In 1906 he was called to take charge of the track elevation of the Atchison, Topeka and Santa Fé Railway through the City of Joliet, Ill.

On completing this work, in 1911, Major Vaughn felt that, after his long career of sixty years of engineering work, he deserved retire-

ment, and he returned to his home in Leavenworth to enjoy the quiet of his remaining years.

His life of sturdy honesty and high principles, together with his marked ability, made a character such as is not often met in these days, and led to an impregnable position in the Engineering Profession.

Major Vaughn was elected a Member of the American Society of Civil Engineers on June 3d, 1891.

CHARLES DOD WARD, M. Am. Soc. C. E.*

DIED JULY 30TH, 1915.

Charles Dod Ward was born on March 15th, 1838, in Williamsburg (Brooklyn), N. Y. He was the son of John Dod Ward and Laura M. Roburds. His father began his career as a machinist, and, by his ability and industry, succeeded in becoming the proprietor of several large iron works and foundries. He was one of the owners of the famous Novelty Iron Works, of New York, and later established the Atlas Foundry, of Jersey City. He took a lively interest in obtaining for New York City a good water supply, and was twice appointed by the Governor of the State a member of the Board of Water Commissioners, under whose direction the Old Croton Water-Works were constructed. Later, he became President of the Commission which built the water-works of Jersey City. Charles Dod Ward, as well as his older brothers John F. Ward, the inventor of the flexible joint for water mains, and the late Lebbeus B. Ward, M. Am. Soc. C. E., inherited from their father a talent for engineering.

Until his sixteenth year Charles Dod Ward attended the Lyceum School in Jersey City. From this school, he went into the Atlas Foundry, owned by his father, to obtain some practical knowledge of foundry work and machinery, but continued his studies for four years under private tutors. Unfortunately, he had attacks of pneumonia during three successive springs, and was advised by his doctors to leave the foundry and lead an outdoor life, as much as possible. In accordance with this advice, he spent two years on farms belonging to a relative and to a friend, taking an active part in the farm work.

Having recovered his health, Mr. Ward decided to become a civil engineer. For two or three years he assisted Mr. Garret J. Van Horne, a well-known civil engineer, of Jersey City, and he also did some work on the Morris Canal in New Jersey.

In his subsequent career, Mr. Ward located and built a great many miles of railroad, and was also engaged on the construction of important water-works. From 1862 to 1865, he made surveys in the State of New York for an extension of the Harlem Railroad, for a railroad from Catskill to Schenectady, and for one from Rondout to Ellensville. In 1866 he was engaged on the location of the Ulster and Delaware Railroad, and soon afterward located part of the Susquehanna and Delaware Railroad, under the late Julius W. Adams, Past-President, Am. Soc. C. E., Chief Engineer.

* Memoir prepared by Edward Wegmann, M. Am. Soc. C. E.

From 1867 to 1869, Mr. Ward was in charge of the construction of the extension of the Utica and Black River Railroad, and then located and had charge of the construction of the Western Division of the New Haven, Middletown, and Willimantic Railroad (New York and Boston Air Line) to December, 1870.

At the request of the late I. C. Buckhout, M. Am. Soc. C. E., Chief Engineer of the Harlem Railroad, Mr. Ward took charge, in 1872, of the surveys for an underground railroad in New York City from the City Hall to the Grand Central Station, which the late Commodore Vanderbilt proposed to build. Complete plans and estimates were made for this project, but it was abandoned on account of the cost involved.

In 1876 and 1877, Mr. Ward was Chief Engineer of a branch railroad in Connecticut from Colchester to the New York and Boston Air Line. Subsequently, he was engaged for about a year on the construction of the Metropolitan Elevated Railroad on Sixth Avenue, New York City. During 1881, he located parts of the T. C. and St. Louis Railroad. In 1886, he was engaged by F. C. Curtis, Chief Engineer of the New York, New Haven, and Hartford Railroad, to make a re-survey of that line in Connecticut from Middletown to Willimantic, with the view of making improvements in the alignment and grades.

Mr. Ward began his experience on water-works in 1866 under the late James P. Kirkwood, Past-President, Am. Soc. C. E., whom he assisted in preparing plans for enlarging the water-works of Cincinnati, Ohio. He was soon afterward engaged for a short time on the water-works of New York under Alfred Craven, M. Am. Soc. C. E., Chief Engineer. In 1871 and part of 1872, he was Principal Assistant Engineer, under the late J. Herbert Shedd, M. Am. Soc. C. E., Chief Engineer, on the water-works of Providence, R. I.

During 1873, he made surveys for the North Hudson Water Commission, of New Jersey, and, in 1883, he was engaged on surveys and plans for the New Jersey Water Supply Commission. From 1889 to 1911, he was heavily interested in the Portsmouth and Suffolk Water-Works, of Virginia, on which he acted as Engineer and in other official capacities. From November, 1894, to May 1st, 1910, Mr. Ward was engaged on the maintenance and extension of the Brooklyn Water-Works, on which he had charge of many important improvements. He resigned his position on account of ill-health, and practically retired from the active pursuit of engineering; but he maintained his interest in the Profession, and occupied his time in studying important engineering problems. One of these was the proposed improvement of the East River at Hell Gate; Mr. Ward proposed a plan for this work, which had considerable merit and was indorsed by a number of persons interested in navigation, to whom it was submitted.

He made two extensive trips abroad. In December, 1874, with a party of friends, he visited Palestine, Egypt, Mount Sinai, Petra, and the east side of the Dead Sea, and traveled much in Europe. His second trip lasted from June, 1878, to December, 1879, and extended from the Mediterranean to the North Cape of Norway and from England to Moscow, Russia.

In 1880, the late Ashbel Welch, Past-President, Am. Soc. C. E., and Mr. Ward were examining a relief map of the proposed route of the Panama Canal, and Mr. Ward suggested that a dam should be built at Gatun. At that time neither of these engineers knew that M. Godin Lepinay had made the same recommendation at the International Congress of Surveys for an Interoceanic Canal, held at Paris, France, in 1879. In discussing the paper by the late A. G. Menocal, M. Am. Soc. C. E., entitled "Inter-Oceanic Canal Projects", in April, 1880,* Mr. Welch stated:

"The first thought of an American canal and river engineer, on looking at M. de Lesseps' raised map, is to convert the valley of the lower Chagres into an artificial lake, some 20 miles long, by a dam across the valley at or near the point where the proposed canal strikes it a few miles from Colon, such as was advocated by Mr. C. D. Ward."

In 1904 Mr. Ward read before the Society a paper on the Gatun Dam,† in connection with the Panama Canal. The Isthmian Commission of 1899 had selected Bohio as the site for a dam to be constructed across the Chagres River. Mr. Ward pointed out in his paper the great advantages that would be derived by building the proposed dam some 10 miles farther down stream at Gatun. He suggested, also, the construction of a dam at La Boca, at the Pacific end of the Canal.

The late Alfred Noble, Past-President, Am. Soc. C. E., a member of the Board of Consulting Engineers appointed in 1905 to report on the plans that should be adopted for the Panama Canal, said, in a letter dated April 18th, 1907, about Mr. Ward's paper on the Gatun Dam:

"The Ward paper created so deep an impression among engineers that one of the earlier examinations made by the First Construction Commission appointed by President Roosevelt was to determine by means of boring the depth to rock and the nature of the overlying earth along the line of the proposed dam. The earth was found to extend to such a depth that the construction of a masonry dam was found to be impracticable, and the project for a dam of this kind was too hastily set aside. There were many, however, who believed with Mr. Ward that a perfectly safe and satisfactory dam could be made of earth, and that the advantages of the location from the point of view of navigation were so obvious that the subject was not allowed to

* *Transactions*, Am. Soc. C. E., Vol. IX, p. 148.

† *Transactions*, Am. Soc. C. E., Vol. LIII, p. 36.

drop, and it received favorable consideration from the minority of the Board of Consulting Engineers in 1905.

"While it is possible to say that the Gatun location would not have been adopted had Mr. Ward's paper not been written, the fact that the first suggestion to this end was dropped and remained unheeded, except by him, for more than twenty years, and that its revival was due to him exclusively, make it most probable that, but for his persistent efforts, the excellent plan of Canal now adopted would not have developed, and either an inferior lock plan or a far more costly sea level plan would have been adopted."

Mr. Ward's paper on the Gatun Dam was presented to the Board of Consulting Engineers of the Panama Canal of 1905, by which Board it was duly considered and ordered reprinted in full in the Appendix of its Report. Both the dams advocated by Mr. Ward have been built, and the cost of the Canal has thereby been much reduced. Although his paper had undoubtedly much to do with the selection of the sites for these two dams, Mr. Ward never received any proper recognition for the great service he had performed by his recommendations therein relative to the Gatun Dam.

As an engineer, he had excellent judgment, both in the location of public works and in their construction, which was greatly appreciated by his superiors. He was exceedingly modest and retiring, and this disposition may have interfered, at times, with his obtaining proper recognition. All those who enjoyed his friendship felt the charm of his fine mind and sterling character.

Mr. Ward was married on December 6th, 1882, to Ada Davey, daughter of Henry L. Davis, of Oswego, N. Y., and widow of William Augustus Davey, of Jersey City, N. J. He is survived by his wife and by a daughter, Mrs. Gerald Stuart O'Loughlin. A second daughter—Mary Frances—died in 1889.

Mr. Ward was elected a Member of the American Society of Civil Engineers on March 3d, 1869.

CHARLES PERKINS WEBBER, M. Am. Soc. C. E.*

DIED JANUARY 30TH, 1916.

Charles Perkins Webber, son of the Rev. Putnam Webber, of North Andover, Mass., was born at Hudson, Mass., on November 18th, 1876. From September, 1898, to June, 1902, he was a student in the Civil Engineering Department at Brown University, but, owing to illness and the loss of a term during his Junior year, he did not receive his degree.

On leaving college Mr. Webber went to Mexico, and, from June, 1902, to June, 1904, was employed on the Mexican Central Railroad as Assistant Division Engineer. He was engaged on the construction of the Lecheria-Sandoval Branch from June to September, 1902; was Division Engineer in charge of the construction of 30 km. of the road from September, 1902, to October, 1903; was Track Engineer on the San Pedro Branch from October to November, 1903; and Topographer, Leveler, and Transitman on the location of the Tampico Short Line, from November, 1903, to June, 1904.

From June, 1904, to August, 1905, Mr. Webber was engaged, as Resident Engineer in charge of 100 km. of reconstruction work, on the Vera Cruz and Pacific Railroad. He then returned to the Mexican Central Railroad as Division Engineer in charge of construction on the Colima Extension, and in April, 1907, was made Assistant Chief Engineer, in charge of proposed branch lines and of the Tampico Short Line.

From January, 1908, to August, 1910, he was employed, as Engineer of Betterments and Additions, with the Vera Cruz and Pacific Railroad, and was then made Chief Engineer in charge of the location and construction of branch lines, etc., of that Company.

In August, 1910, Mr. Webber was appointed Principal Engineer of the Construction Department of the Ferrocarril Nacional de Mexico, and was engaged on the construction of the San Andres Tuxtla Branch until 1913, when he became Chief Engineer of Construction for the Tampico-Panuco Valley Railway Company, Limited. He located and constructed the section of this road from Tampico to Panuco, carrying out the work with conspicuous courage and judgment under very difficult political conditions.

During his career of 14 years of railroad work in Mexico, Mr. Webber won a high reputation, both as a clever engineer and as a precise and honorable worker, fair alike to his companies and to his contractors.

* Memoir prepared by the Secretary from information furnished by Edward Dixon, Esq., General Manager and Engineer, Tampico-Panuca Valley Railway Company, Limited, and from records on file at the Society House.

He was attacked by smallpox, which was prevalent among his men, and which his duty compelled him to face, and died at Panuco, Ver., Mexico, on January 30th, 1916, within a few days of the projected opening of the road he had completed. His death is deeply mourned by his friends and colleagues and by all who knew his loyal and dependable nature.

Mr. Webber was elected an Associate Member of the American Society of Civil Engineers on October 7th, 1908, and a Member on January 7th, 1913.

CHARLES CHANCELLOR WENTWORTH, M. Am. Soc. C. E.*

DIED NOVEMBER 11TH, 1915.

Charles Chancellor Wentworth was a descendant of Colonial Governor Wentworth, of the New Hampshire Colony, and was born in Philadelphia, Pa., on February 21st, 1856. After preliminary schooling at private and public schools and Rugby Academy, he was graduated, in 1876, from the University of Pennsylvania.

In the fall of that year he acted as Levelman on the Seaboard Pipe Line survey from Shippensburg eastward to Baltimore, Philadelphia, and Chester, under Gen. Herman Haupt, Chief Engineer.

In 1877-78, he was employed as Instrumentman on the location and construction of the Foxburg, St. Petersburg and Clarion Railroad, of Pennsylvania, under John Graham, Jr., Chief Engineer.

In 1879, Mr. Wentworth was appointed Assistant Engineer in charge of location of the New River Railroad Company, in Virginia, a line which subsequently became the first outlet of the now well-known Pocahontas coalfield. The construction of this line was carried to completion by the Norfolk and Western Railroad, and, in 1881, Mr. Wentworth was employed on this work under the Chief Engineer of the latter road.

From 1883 to 1889, he was Assistant Engineer to the Chief Engineer of the Norfolk and Western Railroad, with his main office in Roanoke, Va. During that period he was in direct charge of the various engineering construction works, including tidewater and division terminals, branch lines, shops, stations, much bridge work, and other forms of railroad construction, including the improvements in connection with the development of the Pocahontas coalfield.

In 1889, Mr. Wentworth left the service of the Norfolk and Western Railroad to become Chief Engineer of the American Bridge and Iron Company, of Roanoke, Va. While in this position he designed and put into effect a system of standardized highway bridge construction whereby bridges of 200 ft. span or less, including drawbridges, could be constructed, through uniformity of design, with a minimum amount of office work. This system has been used for many years by that Bridge Company and its successor Company.

In 1898, Mr. Wentworth left the American Bridge and Iron Company and re-entered the service of the Norfolk and Western Railway Company as Bridge Engineer. During a number of years he made a study of velocity grades and their effect in the revision of existing track, as governed by the tractive power of a locomotive at varying speeds.†

* Memoir prepared by Charles S. Churchill, M. Am. Soc. C. E.

† The results of these studies were published in the *Railroad Gazette*, December 8th, 1899, *et seq.*

He also made a study of the theory of railroad curves, with a view to establishing positive and practical methods of assuring the easy motion of cars.*

In 1901 he made an analysis of the theory and design of hydraulic rams, in order to extend their application to pumping large quantities of water. For this he was awarded the Longstreth Medal by the Franklin Institute on December 3d, 1902.

In the same year, in collaboration with the writer, he made a considerable detailed study of the delivery of fresh air through tunnels, the first application of which was made to Elkhorn Tunnel, on the Norfolk and Western Railway, which method afterward became known as the Churchill-Wentworth System of Tunnel Ventilation.†

On February 1st, 1903, Mr. Wentworth was appointed Principal Assistant Engineer of the Norfolk and Western Railway, which position he held until his death.

He was a very active member of the American Railway Engineering Association, and served on various committees thereof. He was in generally good and vigorous health until suddenly overtaken by death while asleep. He is survived by his widow, Sophia Park Wentworth, three sons, and three daughters.

Mr. Wentworth was elected a Member of the American Society of Civil Engineers on April 4th, 1888.

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* "Line and Surface for Railway Curves", *Transactions*, Am. Soc. C. E., Vol. XLVIII, p. 357.

† The formulas have been given by Mr. Wentworth in *Transactions*, Am. Soc. C. E., Vol. LIV, Part C, p. 567.

JAMES KNAPP WILKES, M. Am. Soc. C. E.*

DIED JANUARY 8TH, 1916.

James Knapp Wilkes was born in Danbury, Conn., on August 21st, 1859. He was the only son of Matthew and Henrietta Knapp Wilkes, whose ancestors settled in Connecticut in the Seventeenth Century.

After he was graduated from the Danbury High School in 1876, he took up the study of scientific and engineering subjects under private tutors for two years, and in 1880 was graduated from the State Normal College at Albany, N. Y. During the same year Mr. Wilkes entered the office of Mr. D. G. Penfield, of Danbury, Conn., and was engaged in general engineering work in that vicinity, including the construction of the New York and New England Railroad between Danbury, Conn., and Hopewell Junction, N. Y.

In 1884 he opened an engineering office, and engaged in private work; he also performed all the engineering work required for the Borough of Danbury.

In 1887 Danbury became a city, and, in addition to his private duties, Mr. Wilkes was appointed City Engineer, having charge of all the municipal work, including regulating, grading, and paving streets, the construction of sewers, and the extension of the water supply, together with the distribution system and the enlargement of the dams and reservoirs. In addition to his duties for the City of Danbury, he also constructed and designed a dam and pipe line near Bethel, Conn., besides being engaged as expert on legal questions arising from the development of water power in that vicinity.

In 1890 Mr. Wilkes went to New Rochelle, N. Y., as Assistant Engineer in charge of the construction of sewers which had been designed and were being constructed under the direction of the late Horace Crosby, M. Am. Soc. C. E.

In 1891 he was again called to Danbury to take up the work of City Engineer, and he held that office until 1893, when he was appointed Chief Engineer of the Sewerage Commission of New Rochelle, N. Y., which position he held for 6 years, having charge of the design and extensions of the sewers and storm-water drains, as well as a sewage purification works, by chemical precipitation, for taking care of the sewage from the westerly half of the city. Owing to the rapidly increasing growth of the city, it became necessary to remove the original eastern outlet, in order to prevent the pollution of the shores and bathing beaches. Mr. Wilkes solved the problem by carrying the sewage through a tunnel driven under Hudson Park and a submerged pipe under the harbor, 3 000 ft. into Long Island Sound. In

* Memoir prepared by C. W. S. Wilson, M. Am. Soc. C. E.

addition to this sewerage construction, he also had charge of the maintenance and repairs of the sewers and drains throughout the city.

In 1899 Mr. Wilkes was engaged by the Metropolitan Street Railroad in the construction of a double-track electric system through 42d Street, New York City.

In 1900 he was employed by The Hoffman Engineering and Contracting Company, as Engineer in the construction of the foundations for the new cadet quarters at the United States Naval Academy, Annapolis, Md.

In 1901 he was Superintendent for Mr. W. A. Engeman, on the construction of a railroad from the Connecticut River to Benvenue granite quarries and the erection of cableways at quarries near Middletown, Conn.

In 1902 Mr. Wilkes was appointed City Engineer of New Rochelle, N. Y., which position he held until 1912, having charge of all engineering matters pertaining to the city, the work consisting principally in the improving and paving of streets, extending sewers, and building a 30-in. inverted siphon sewer under Echo Bay. In 1912 he resigned as City Engineer and formed the Wilkes-Casey Engineering and Contracting Company, of which he was President, carrying on general construction work for municipal and private operation, the Company's work being confined principally to the States of New York and Connecticut.

Mr. Wilkes was taken sick on January 1st, 1916, and died of pneumonia at his home in New Rochelle on January 8th, 1916. In 1885 he was married to Capitola Baker, of Patterson, N. Y., who, with one daughter, survives him.

Mr. Wilkes was elected an Associate Member of the American Society of Civil Engineers on June 3d, 1891, and a Member on April 3d, 1906. He was also a Charter Member of the Connecticut Society of Civil Engineers.

DAVID WILLIAMS, M. Am. Soc. C. E.*

DIED NOVEMBER 27TH, 1915.

David Williams, the son of Griffith and Sinal Williams, was born in Holyhead, North Wales, on February 6th, 1854. When still an infant he was taken by his mother to Richmond, Que., Canada, where his father was Engineer of Construction on what is now the Grand Trunk Railway. Mr. Williams attended the schools in Richmond and, later, St. Francis College, afterward serving as Lieutenant in the Light Artillery and being stationed in the Citadel at Quebec.

From 1874 till 1882, Mr. Williams was associated with Mr. H. C. Cleveland, a prominent Railway Locating Engineer of the Eastern Townships. In 1882, he joined the Engineering Staff of the Canadian Pacific Railway, being assigned to the Northwest Lake Superior Division, on which Division he remained until 1886. During his employment with the Canadian Pacific Railway, he had charge of the location of nearly 200 miles of track around the Great Lakes. In 1886, he was appointed Chief Engineer of the Drummond County Railway, now a part of the Intercolonial Railway System, and resigned from that position in 1887 to accept the office of Locating Engineer of the Hereford and Upper Coos Railroad, now a part of the Maine Central Railroad System. After completing his work as Locating Engineer, Mr. Williams had charge of the construction, and built nearly 100 miles of road into the extensive lumber regions in the Eastern Townships of Quebec.

In March, 1890, he was appointed Road Master of the Western Division of the Boston and Maine Railroad. In 1891, he was made Assistant to the Chief Engineer, which position he held until 1893, when he was appointed Division Engineer. He served as Division Engineer from 1893 until he retired from railroad engineering in 1911. During his connection with the Boston and Maine Railroad Mr. Williams had charge of the double-tracking of the Vermont Valley Railroad and the Sullivan County Railroad. After his resignation from the Boston and Maine Railroad, he had charge of the separation of grades on the Revere Beach and Lynn Railroad. In 1912, he was appointed Chief Engineer of the E. and T. Fairbanks and Company's plant, and had entire charge of the construction of the extensive additions to its home plant.

At the time of his death at St. Johnsbury, Vt., on November 27th, 1915, Mr. Williams was employed by the State of Vermont as Consulting Engineer to the Public Service Commission and the State Highway Commission.

* Memoir prepared by R. Q. Hamilton, Asst. Engr., Boston and Maine Railroad, St. Johnsbury, Vt.

Mr. Williams was a man of sterling qualities. His judgment was sound and practical, and given only after careful study and deliberation, and he was frequently called on as an expert engineer in litigation. To those who knew him best, he was a man whose friendship was most highly valued. Although by nature of a retiring disposition, when he was among those whom he loved, his reminiscences were a veritable well of knowledge.

On February 10th, 1891, he was married to Miss Lucina Mackay, of Cookshire, Que., Canada, who survives him.

Mr. Williams was elected a Member of the American Society of Civil Engineers on May 4th, 1898.

FRANK ROBERT WILLIAMSON, M. Am. Soc. C. E.*

DIED JULY 11TH, 1915.

Frank Robert Williamson was born in Harford County, Maryland, on December 22d, 1867. In his early boyhood his parents moved to Illinois and settled in Clinton County.

He attended a country school and the graded school in the Village of Huey, and prepared himself for teaching by attending the Normal School at Carbondale.

In the fall of 1887 he entered the University of Illinois, and, after his sophomore year, he taught a country school for a year and then returned to the University, where he completed the course and was graduated in 1892 with the degree of B. S. in Civil Engineering.

On leaving the University Mr. Williamson entered the employ of the Pittsburgh Bridge Company, and served in both the Pittsburgh and Chicago offices of the Company as Draftsman and Checker.

In 1895 he was in the office of Ralph Modjeski, M. Am. Soc. C. E., in Chicago, where he assisted in designing the draw-span of the Chicago, Rock Island and Pacific Railway across the Mississippi River at Rock Island, Ill.

During 1896 he was employed in the office of the Wisconsin Bridge Company, at Milwaukee, Wis., as Draftsman and Checker.

Mr. Williamson was in the office of the Bridge Division of The Sanitary District of Chicago from 1897 to 1899, where he computed the stresses and designed the sections of eight of the draw-spans over the Main Drainage Channel of the Sanitary District.

He was Assistant Engineer in charge of the drafting-room of the Bridge Department of the Chicago and Northwestern Railway during 1900 and 1901. Under the direction of the Engineer of Bridges, he designed the double-track viaduct across the Des Moines River near Boone, Iowa.

Mr. Williamson was in charge of the drafting-room of the Scherzer Rolling Lift Bridge Company, at Chicago, from 1901 to 1908, and assisted in the design of many important bascule bridges of the rolling lift type. Many improvements in detail and design were due to his skill.

In February, 1908, he again entered the office of The Sanitary District of Chicago, where he continued his work until the time of his death. He assisted in the preparation of the plans for the Eight-Track Bridge across the Main Drainage Channel of the Sanitary District, and was Resident Engineer in charge of the construction

* Memoir prepared by Samuel T. Smetters, M. Am. Soc. C. E.

of the bridge. His last important work was in connection with the design and construction of the bascule bridge across the Chicago River at Jackson Street. This bridge was not completed at the time of his death.

Mr. Williamson was a man of few words, and was very energetic and determined. He was of uncompromising honesty, and could be trusted with any responsibility he would accept or which he considered his duty, with entire confidence that the interests of his employers would be conserved and the best results obtained.

Mr. Williamson was ever ready to listen with patience and forbearance to the views of others, but was tenacious of his own convictions, and, being gifted with sound judgment, his opinion always commanded respect. His associates and assistants held him in high esteem for his ability and good qualities; he was respected by contractors for his untiring effort to obtain the best work and for his good judgment and timely advice often rendered.

He won the esteem and respect of all who knew him, and always stood for the highest and best interests of the community in which he lived. He was a member of the Normal Park Methodist Episcopal Church.

He is survived by his widow and two sons, Frank Martin and Edwin Paul, aged 14 and 10 years, respectively.

Mr. Williamson was elected a Member of the American Society of Civil Engineers on November 7th, 1906.

ARTHUR FRANCIS WROTNOWSKI, M. Am. Soc. C. E.*

DIED OCTOBER 28TH, 1911.

Arthur Francis Wrotnowski, the son of Stanislaus Wrotnowski and Catharine Ozibulska Wrotnowski, was born on October 14th, 1843, at Clermont, France, where his father was then a beet sugar manufacturer. When he was eleven years old, he came to the United States with his parents who settled at Baton Rouge, La., where his father had a cane-sugar mill.

During the Civil War (1862) he enlisted in the Federal Army and was made Lieutenant-Colonel of an Engineering Corps stationed in Texas during the latter years of the war.

In January, 1868, Col. Wrotnowski was married to Miss Josephine Rachel Thomas, of Philadelphia, Pa.

During the construction of the jetties at the mouth of the Mississippi River, he served on the Board of State Engineers of Louisiana and, in 1876, he was appointed Chief State Engineer of that Board. He was also Supervising Architect of the Custom House at New Orleans, La., which was being remodeled, from 1878 to 1881.

During 1882-83, he was in charge of the harbor works at Vera Cruz, Mexico, under a French company which discontinued the work there in 1883.

Mrs. Wrotnowski died in May, 1883, and in the following year, Col. Wrotnowski having invested heavily in Florida, made his home there, in order to look after his investments. He founded the Town of Clermont, called after his birthplace in France.

In February, 1885, he was married to Miss Angie Bladen, of Philadelphia, Pa.

In 1887-88 he was connected with the East Coast Canal Company of Florida and, in 1889, went to Tampico, Mexico, as Resident Engineer in charge of the construction of the jetties at the mouth of the Panuco River, which were being built by the Mexican Central Railway and of which the late E. L. Corthell, President, Am. Soc. C. E., was Consulting Engineer in charge. These works were completed in 1892, and Col. Wrotnowski then opened an engineering office in New Orleans, La.

In 1895 he had charge, for a few months, of the harbor works of Vera Cruz, Mexico, then being built by Sir Weetman Pearson, now Lord Cowdrey.

About 1897, he removed to the western coast of Mexico and settled at Guaymas and Hermosillo, Sonora. He constructed a number of

* Memoir prepared by the Secretary from information on file at the Society House.

buildings in these two cities, among which were the State Penitentiary, markets, bank, etc., etc.

Col. Wrotnowski's declining years were marred by prolonged ill health, and he only survived his second wife by about 6 months. He died on October 28th, 1911, in Nogales, Ariz., where he had gone for his health, and is buried there. He is survived by one daughter by his first marriage, Mrs. C. F. de Ganahl, of New York City, and by two daughters and a son, by his second marriage, Mrs. Van Archibald Smelker and Mrs. William Wells Griffith, both of Nogales, Ariz., and Arthur Corthell Wrotnowski, of San Diego, Cal.

Col. Wrotnowski was elected a Member of the American Society of Civil Engineers on July 12th, 1877.

WILLIAM FREDERICK ALLEN, Assoc. M. Am. Soc. C. E.*

DIED NOVEMBER 9TH, 1915.

William Frederick Allen was born in Bordentown, N. J., on October 9th, 1846, the son of Col. Joseph Warner Allen and Sarah Burns (Norcross) Allen. On the paternal side the family is of English origin. Samuel Allen, the founder of the American line, came from Chew Magna, near Bristol, England, in 1681, and settled in the Colony of Pennsylvania, at Bridgewater, near Bristol. Mr. Allen's father was a civil engineer, and had charge of many water-power developments and railway enterprises. At the beginning of the Civil War he was Deputy Quartermaster-General of New Jersey and, later, raised the 9th Regiment, New Jersey Volunteers, of which he was Colonel. He served under Gen. Burnside in the North Carolina expedition, and met his death by drowning off Cape Hatteras, on January 15th, 1862.

Mr. Allen was educated at the Bordentown Model School and at the Protestant Episcopal Academy in Philadelphia. His engineering experience began in 1862, as Rodman with the Engineer Corps of the Camden and Amboy Railroad. In the following year he became Assistant Engineer. In 1868 he was transferred to the West Jersey Railroad as Resident Engineer, in which capacity he served nearly five years. At this period (1870), he founded and laid out the Town of Wenonah, N. J. In October, 1872, he became Assistant Editor of the *Official Railway Guide*, and six months later was made Editor and Manager of the National Railway Publication Company, of which he was elected Vice-President in 1910 and President in 1914.

Some idea of the enormous growth of the *Official Railway Guide*, published by this Company, which contains complete time-tables, accurately corrected to date every month, of all the railways of the United States, Canada, and Mexico, may be gathered from the fact that when Mr. Allen took charge in 1872, it had only 192 pages, but has now increased to more than 1 600 pages, and is the largest monthly publication in the world. It is admirably arranged and indexed for ready reference, containing an authorized and complete list of railway officers forming the managing staff of each railroad company, also railway maps, mileage operated, editorials, official news, and other information of value to the railway public. It is in use the country over as the standard reference authority in the transportation service.

In addition to his duties connected with the National Railway Publication Company, Mr. Allen had been Secretary of the American Railway Association and its predecessors since April, 1875, when he

* Memoir prepared by Charles A. Hammond, M. Am. Soc. C. E.

was appointed Secretary of the General Time Convention, and in October, 1877, of the Southern Time Convention. In April, 1886, these organizations were consolidated to form the American Railway Association, and Mr. Allen continued as Secretary, becoming, in June, 1909, its General Secretary and Treasurer, which positions he held until his death.

The membership of the American Railway Association is composed of the railroad companies in the United States, Canada, and Mexico, operating 300 000 miles of railway. The object of the Association is the recommendation of methods for the management and operation of American railways. It has perfected the "Standard Code of Train Rules, Block Signal Rules and Interlocking Rules"; the improvement of Car Service Methods, including the Code of Per Diem Rules looking to increasing the efficient distribution of freight cars; a National Code of Car Demurrage Rules, to facilitate the prompt movement of freight cars; Standard Track Scale Specifications and Rules, and Uniform Rules for the Weighing and Re-weighing of Carload Freight; the adoption of Standard Heights for Draw-bars, also of Standard Dimensions for Box Cars and for Rail Sections; and a Bureau for the Safe Transportation of Explosives.

In his capacity as Secretary of the Association, Mr. Allen became intimately acquainted with a large number of railway managers. The Presidency of the Association has been held successively by different men, from different parts of the country, but the Secretaryship has been a permanent feature, and his administration of the office has been an important element in the Association's prosperity, contributing in no small degree to the remarkable success of its efforts in standardizing and improving American railway practice.

Undoubtedly, Mr. Allen's greatest claim to a world's gratitude is his monumental achievement in the establishment of the present system of "Standard Time". In 1881, the General Time Convention referred to him for solution the difficult problem of working out a standard of time reckoning that would obviate the confusion resulting from the use of the fifty or more local-time standards then prevailing in the United States. His report was submitted in 1883. His plan divided the country into five even-hour zones, based on the quadrant meridian (90° west of Greenwich) and the meridians of each 15° east and west of it. The report also provided for an elastic boundary line between the hour zones, instead of a strictly longitudinal division, and in its details fixed every point at which the hour-change was to be made, worked out the method of passing, without confusion, from one hour-standard to the other, adjusted the difference between local and standard time, and embodied every practical provision for placing the system in immediate effect. The report was unanimously endorsed

by the Association, and Mr. Allen was empowered to secure its adoption by the railways. In this work, he had the co-operation of the Cambridge and the National Observatories.

The change in the operating time-tables of the many different railroads throughout the country was made at noon, eastern time, on Sunday, November 18th, 1883, without delay or disturbance. For this splendid achievement, pronounced one of the greatest in the history of transportation, Mr. Allen was elected to Honorary Membership in many American and foreign scientific societies, and received, in 1906, from Princeton University, the honorary degree of Master of Science.

The American system of standard time, successfully inaugurated through Mr. Allen's untiring efforts and special facilities for reaching and influencing the railway managements of the country and other interests involved, has now been adopted by all the nations of Europe, by Japan, and by the South American countries—practically by all the civilized nations of the world.

Mr. Allen was a delegate of the United States Government to the International Meridian Conference in 1884, and to the International Railway Congress at Paris in 1900. He was a delegate of the American Railway Association to the International Railway Congresses at London, 1895; Paris, 1900; Washington, 1905; and Berne, 1910. Since 1910, he had been a member of the Permanent Commission of the Congress Association. In 1905, he had charge of all the arrangements for the session of that year at Washington, which was presided over by Mr. Stuyvesant Fish, and considered one of the most successful meetings in the history of the organization. At this session Mr. Allen was Associate Secretary-General. For his services at the Washington session of the Congress, he received from the Belgian Government the decoration of Chevalier of the Order of Leopold.

Mr. Allen was also greatly interested in efforts to establish the metric system in the United States, and was Chairman of the Committee of the American Railway Association on this subject.

The prodigious and versatile activity of this remarkable man may be appreciated when we remember his affiliation with many different organizations in the railway, scientific, business, and social world. Besides being President of the National Railway Publication Company, and of the Knickerbocker Guide Company, he was Vice-President of the New York Transfer Company, and a Director in numerous other corporations; a member of the Geographical Society of Vienna, Austria; Geographical Society of Lima, Peru; American Geographical Society; National Geographic Society; American Metrological Society; American Academy of Political and Social Science; American Statis-

tical Association; American Economic Society; American Association for the Advancement of Science; American Forestry Association; American Railway Guild, of which he was a Past Master and principal founder; Loyal Legion; New Jersey Historical Society; Washington Academy of Sciences; the New England Society of Orange, N. J., of which he was ex-Counsellor; and the Pennsylvania Society of New York.

Mr. Allen had resided at South Orange, N. J., since 1881, and had taken an active part in the development of that residential community, serving on the Drainage Commission to straighten the Rahway River, and on the Board of Trustees of the Village. He was Warden and, for many years, a Vestryman of the Church of the Holy Communion in South Orange. He was also a member of Century Lodge and a Trustee of the South Orange Free Public Library.

On Sunday, November 21st, 1915, a public meeting was held in Columbia School Hall, South Orange, to express the appreciation of the citizens of the village for Mr. Allen's public work, as a citizen as a friend, and as a neighbor. More than 600 were in attendance, and addresses were made by Spencer Miller, M. Am. Soc. C. E., Member of the Naval Advisory Board; Edward D. Duffield, Ex-Assistant Attorney-General, State of New Jersey; and Wilson Farrand, Headmaster, Newark Academy.

In 1889, with his associates, Mr. Allen organized and became President of the Meadow Land Society, with the object of preserving the beauty of the valley in the center of South Orange Village, and of providing facilities for field sports. The Society acquired the large tract of land which made possible the fine grounds of the South Orange Field Club, of which organization he was also a charter member, and for many years one of the Board of Governors, and President. A part of the ground formerly owned by the Meadow Land Society is now Cameron Field, and, in the creation of this public playground, Mr. Allen was also an active participant and one of the Trustees. In 1908, he was a delegate of the Eighth New Jersey Congressional District to the National Republican Convention in Chicago.

His clubs include the Railroad and the Traffic Clubs of New York; the South Orange Field and South Orange Republican Clubs, of South Orange, N. J., of both of which he had been President; and the Essex County Country Club.

Mr. Allen was married on April 20th, 1871, to Caroline Perry Yorke, daughter of Hon. Thomas Jones Yorke, of Salem, N. J., who survives him, together with four sons—all graduates of Princeton University—Yorke Allen, a lawyer of New York City, Frederick Warner Allen, connected with the banking firm of Wood, Struthers and Company, of New York, Eugene Yorke Allen, Assoc. M. Am. Soc. C. E., Civil

Engineer with the Central Railroad of New Jersey, and John Sinnickson Allen, Secretary of the Gamewell Fire Alarm Telegraph Company.

Mr. Allen was one of the most lovable of men. His intercourse with his fellows was always marked by kindness, sincerity, and a beautiful courtesy. Energy, faithfulness, and a conscientious discharge of duty were his prominent characteristics, and the editorial pages of the *Guide* reveal his fine and discriminating intellect. The burden of his multifarious activities and responsibilities was borne with such grace and apparent ease, that few realized the strain it must have been, until Nature at length gave way. It was while motoring to witness the Princeton-Harvard football game, on Saturday, November 6th, that he suffered an apoplectic attack, becoming unconscious before home could be reached, and lingering painlessly in that condition until his death on Tuesday, November 9th. The funeral services, held in the church he loved so well, were attended by a large number of railroad men, for whom a special train had been furnished by the Delaware, Lackawanna and Western Railway Company.

Mr. Allen had made all his preparations for attending the fall meeting of the American Railway Association, held in Chicago, on November 17th, 1915, at which meeting the following action was taken:

"In Memory of William Frederick Allen

"Secretary of The American Railway Association from the date of its organization, April, 1886, to November 9th, 1915; General Secretary and Treasurer from June, 1909, to November 9th, 1915; Secretary of the General Time Convention from April, 1875, and Secretary of the Southern Railway Time Convention from October, 1877, until their consolidation in 1886, at which time The American Railway Association was created; originator of the idea of Standard Time, which has been universally adopted.

"Mr. Allen's industry and loyal co-operation with his associates during a period of forty years were always most helpful and welcome to them, and they record with profound sorrow his death at his home in South Orange, New Jersey, on the afternoon of November 9th, 1915, in the seventieth year of his age.

"His great ability, high character, and forceful and amiable personality endeared him to all who knew him, and inspired their admiration, friendship, and esteem. In his death the Association has lost an able and resourceful officer, and his friends and associates deeply mourn the loss of a friend and counsellor.

"*Resolved*, That this tribute to his memory be inscribed on the minutes of this Association, and that an engrossed copy be delivered to his family, to whom The American Railway Association tenders its sympathy in their great sorrow."

Mr. Allen was elected an Associate Member of the American Society of Civil Engineers on January 3d, 1909.

GEORGE LENOX CRAWFORD, Assoc. M. Am. Soc. C. E. *

DIED FEBRUARY 3D, 1916.

George Lenox Crawford was born near Napoleon, Henry County, Ohio, on June 23d, 1864. He attended the public schools, and, during vacations, assisted his father in drainage engineering, etc., which fact probably influenced his choice of a profession.

In June, 1886, Mr. Crawford entered the service of the Union Pacific Railroad as Chainman and Rodman, and, from that time until September, 1890, he was employed as Rodman, Transitman, Assistant Engineer, and Field Engineer, with the Tremont, Elkhorn and Missouri Valley Railroad, the St. Louis and San Francisco Railway, the New Orleans and Northwestern Railway, and the Springfield (Mo.) Electric Street Railway, respectively.

In September, 1890, he began the work with which he was identified during the latter part of his professional life, namely, coal-mining engineering, as Principal Assistant to the Chief Engineer of the Kansas and Texas Coal Company, at St. Louis, Mo. In this capacity, Mr. Crawford had charge of surveys and of the design and installation of coal-mining and coal-handling plants.

From September, 1892, to 1894, he was engaged in private practice as Consulting Engineer on mining and irrigation work at Springfield, Mo., and also at Gunnison, Colo.

In November, 1894, Mr. Crawford returned to railroad engineering as Engineer of the Bridge and Building Department of the Colorado and Southern Railway, at Pueblo, Colo., which position he retained until November, 1895, when he again took up private practice.

From November, 1897, to July, 1898, Mr. Crawford, as Chief Engineer, made surveys, plans, and estimates for a double-track, electric and steam railroad for the Kansas City-Leavenworth Electric Railway, Power and Mining Company of Leavenworth, Kans., but the project was never carried out. In August, 1898, he went to Wyoming as Assistant Engineer on the construction of the new steel main line of the Union Pacific Railroad from Granger to Piedmont, and from January to June, 1899, he served as Transitman on location and in charge of the construction of the tunnel and bridges on the La Veta Cut-Off of the Denver and Rio Grande Railroad.

In June, 1899, Mr. Crawford returned to the practice of coal-mining engineering, in which, except during his term as City Engineer of Grand Junction, Colo., he continued until his death. From June, 1899, to January, 1905, he was Chief Engineer of the Northern Coal and

* Memoir prepared by the Secretary from material furnished by A. L. Fellows, M. Am. Soc. C. E., supplemented by data on file at the Society House.

Coke Company at Denver, Colo., in charge of mine surveys, the design and construction of coal-mining plants, and in making estimates and reports on coal properties, etc. From that position, he went, as Chief Engineer, to the Rocky Mountain Fuel Company, and, in May, 1907, he was appointed General Superintendent and Chief Engineer of the Tam O'Shanter Montezuma Mines and Development Company. For this latter Company, he designed a hydro-electric power plant and an aerial tramway and terminals, and had charge of the construction of a 50-ton concentrating mill.

In February, 1908, Mr. Crawford again took up consulting work, which he continued until April, 1909, when he was elected City Engineer of Grand Junction, Colo., which office he held until 1913. While in this position he designed and constructed sewers, pavements, and a water-supply line for that city. In 1913, he again returned to private practice as a Consulting Engineer with headquarters at Denver, Colo., until 1914, when he was appointed General Superintendent of The Pike's Peak Fuel Company, at Colorado Springs, Colo., which position he held until his death, on February 3d, 1916, from cerebro-spinal meningitis following an attack of pneumonia.

From his boyhood, Mr. Crawford had been a hard worker. Owing to the fact that his father died while he was still young, he had to forego any college or technical training except that which he obtained through his own efforts and studies; nevertheless, he was well educated in the best sense of the word.

He was a man of high moral character, modest and unassuming, but efficient and thorough, and a true friend once his friendship was won. His honor was his first and highest concern.

He was a member of the Plymouth Congregational Church, and a Mason, having been a member of Union Lodge No. 7, of Denver, Colo.

The following resolutions on Mr. Crawford's death, which were adopted by the Colorado Association of Members of the American Society of Civil Engineers, of which he was a member, were presented before the meeting of that Association on February 12th, 1916:

"*Resolved:* That we, the members of the Colorado Association of Members of the American Society of Civil Engineers, deeply deplore our loss, recognizing at the same time in his removal a loss not only to ourselves, but to the community at large; and, further,

"*Resolved:* That this statement and these resolutions be spread upon the records of this organization and that a copy thereof be sent to his family with an expression of our most sincere sympathy."

Mr. Crawford was elected an Associate Member of the American Society of Civil Engineers on September 3d, 1912.

THOMAS HOVENDEN, Jr., Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 19TH, 1915.

Thomas Hovenden, Jr., son of Thomas Hovenden, the well-known artist, and Helen C. Hovenden, was born at Plymouth Meeting, Pa., on March 11th, 1882. His early education was received at the Friends' School, at Plymouth Meeting, and the Friends' Select School, in Washington, D. C., from which he was graduated in 1899. He then entered the Civil Engineering Department of the University of Pennsylvania, and was graduated in 1903 with the degree of B. S.; and received that of M. S. in 1904.

For a year after his graduation Mr. Hovenden held the position of Instructor in the Civil Engineering Department of the University of Pennsylvania, leaving, in 1905, to enter the employ of W. W. Lindsay and Company, Engineers and Contractors, of Philadelphia. From that time until his death, he remained with this firm (whose business is steel-plant and mill-building construction), and rose from a subordinate position to that of General Superintendent, in charge of all construction work done by the Company.

During this time Mr. Hovenden had charge of a great variety of work in steel, concrete, and timber construction. He built several open-hearth and heating furnaces and manufacturing plants of various kinds in the Eastern States. For some months he was in Cuba in charge of the erection of a number of highway bridges for the Cuban Government, for which his firm had the contract.

His sterling character and his ability, which was of the highest order, were much appreciated by his employers and by all with whom he came in contact, and his untimely death, which occurred on September 19th, 1915, at the age of thirty-three, cut short a career of much promise.

Mr. Hovenden was elected an Associate Member of the American Society of Civil Engineers on July 9th, 1912.

* Memoir prepared by W. W. Lindsay & Co., Harrison Bldg., Philadelphia, Pa.

LOREN EDWARD HUNT, Assoc. M. Am. Soc. C. E.*

DIED JANUARY 9TH, 1916.

Loren Edward Hunt was born at Austin, Minn., on January 7th, 1870. Three years later his parents moved to Santa Barbara, Cal., where he attended the Grammar and High Schools. On the completion of his primary education, he entered the University of California, and was graduated from the College of Civil Engineering in May, 1893.

From May, 1893, to July, 1895, Mr. Hunt was Assistant Instructor, and from July, 1895, to April, 1902, Instructor, in Civil Engineering at his Alma Mater, and was in charge of the Laboratory for Testing Materials of Construction during the latter period.

In April, 1902, he entered the service of the Bureau of Engineering of San Francisco, Cal., as Assistant City Engineer in charge of the construction of pavements and sewers. He resigned this position in March, 1903, to accept the direction of the Testing Laboratory of the United States Forest Service, at the University of California, which position he held in conjunction with the Chair of Lecturer in Civil Engineering.

In 1909, Mr. Hunt again returned to the office of the City Engineer of San Francisco, and, in 1911, he was made Chief Assistant City Engineer. This position he held at the time of his death, which took place at his home in San Francisco, on January 9th, 1916, after a brief illness.

In his dealings with the general public and with the contractors transacting business with the City Engineer's Office, Mr. Hunt gained a reputation for the most scrupulous fairness, and gave the best of his many talents, energy, and vitality, even at the sacrifice of his personal health, to the service of the City. He never allowed any personal consideration to conflict with his high sense of public duty, and he was revered as a man as greatly as he was respected as an engineer.

He was a member of the American Society of Testing Materials, the Technical Society of the Pacific Coast, and the Bohemian Club of San Francisco.

Mr. Hunt was elected an Associate Member of the American Society of Civil Engineers on June 3d, 1903.

* Memoir prepared by E. F. Haas, H. I. Randall, and M. M. O'Shaughnessy, Members, Am. Soc. C. E., a Committee of the San Francisco Association of Members of the American Society of Civil Engineers.

JAMES BLAINE MILLER, Assoc. M. Am. Soc. C. E.*

DIED MAY 7TH, 1915.

James Blaine Miller, the son of Thomas C. Miller, was born at Erie, Pa., on October 30th, 1883. He was graduated from the High School at that place in 1899, and from Oberlin College in 1903, with the degree of Bachelor of Arts.

During his last year at college, Mr. Miller passed the Civil Service examinations of the United States Coast and Geodetic Survey, and immediately on his graduation in June, 1903, entered that service as Aid.

From July, 1903, to September, 1906, as Member and Chief of Party, Mr. Miller was engaged on geodetic work in Alaska, Porto Rico, and in various parts of the United States. On his promotion to Assistant, in September, 1906, he was made Chief of Party, in command of the Coast and Geodetic Survey Steamer *Endeavor*, and spent several seasons in charting the unsurveyed waters of the Alaskan Coast and the Hawaiian Islands.

In 1908, Captain Miller was transferred to the Philippine Islands, and, in 1912, during a leave of absence, he spent some time in geodetic work for the Federal Government of Australia. His last cruise was of two years duration and it was near the end of this cruise that he assisted in the rescue of the crew of the Revenue Cutter, *Tahoma*, which was wrecked on September 20th, 1914, among the Aleutian Islands. In a memorial presented to him by the men of the Revenue Cutter Service on that occasion the following is engraved:

"Captain Miller was the first to answer the distress call of the *Tahoma* and navigated his vessel with all possible speed to the scene of the wreck. He conducted a most thorough search for the small boats of the *Tahoma*, traversing dangerous and uncharted water both day and night until the last man was rescued."

In April, 1915, Captain Miller received leave of absence and sailed for Europe on the *Lusitania*. On July 20th, 1915, his body was recovered on the west coast of Ireland and after identification, was removed to Erie, Pa., where funeral services were held in December, 1915.

Captain Miller was elected an Associate Member of the American Society of Civil Engineers on April 2d, 1913.

* Memoir prepared by Walter J. Ryan, Assoc. M. Am. Soc. C. E.

LUDLOW VICTOR CLARK, Assoc. Am. Soc. C. E.*

DIED DECEMBER 28TH, 1915.

Ludlow Victor Clark was born in Chatham, N. Y., on June 6th, 1863. He had been connected with the cement business in Philadelphia, Pa., for nearly thirty years, and, at the time of his death, was Second Vice-President of the Lawrence Portland Cement Company.

He was a man of unusual attainments and of a most attractive personality. He was an extensive traveler and a student of economic and industrial conditions, and the nature of his business made him keenly appreciative of all problems relating to the science of constructive engineering.

Mr. Clark was of that splendid type of character that compels respect and confidence, possessing a fine judgment for delicate action when business problems necessitated the arts of diplomacy and adjustment. He was very precise as to detail, but broad in dealing with subjects as a whole. His great ability was clearly recognized, and, in his death, the cement industry has suffered the loss of a good man. Loyalty and friendship were always the unwavering standards of his conduct, and his associates and friends will feel his loss deeply.

Mr. Clark is survived by a widow and one daughter. He was a member of the Union League Club of Philadelphia, a Mason of high degree, and a Noble of the Mystic Shrine.

Mr. Clark was elected an Associate of the American Society of Civil Engineers on October 4th, 1892.

* Memoir prepared by the Lawrence Portland Cement Company, Philadelphia, Pa.

WALTER COX BOWEN, Jun. Am. Soc. C. E.*

DIED MAY 8TH, 1915.

Walter Cox Bowen was born at Shiloh, N. J., on November 17th, 1886. He received his elementary education in the public schools of Shiloh, and, after his graduation from the High School, taught school in Southern New Jersey for several years, prior to entering college.

Mr. Bowen began his professional life in 1908, when he entered Rutgers College as a student in Civil Engineering. During the four years he attended college, he spent his summer vacations in the field on various engineering projects.

After his graduation in 1912, Mr. Bowen entered the engineering offices of Clyde Potts, M. Am. Soc. C. E., as Assistant Engineer, where he remained until his death. While he was in charge of the construction of sewage and water works for Milltown, N. J., a suburb of New Brunswick, he took post-graduate work in chemistry and bacteriology at Rutgers College, in order to prepare himself for his chosen profession in sanitary engineering.

He died on May 8th, 1915, as a result of injuries received in an explosion in a sewage plant at Ocean Grove, N. J.

While at college Mr. Bowen took a very active part in athletics and social life, and was a member of Delta Kappa Epsilon Fraternity. In his Senior Year he was Manager of the football team, Captain of the baseball team, Captain of one of the companies in the Cadet Corps, and President of the Civil Engineering Club. He also belonged to the Masonic Fraternity.

Nothing more apt could be said at this time than to quote the closing words of the sermon delivered by President Demarest, of Rutgers College, at the funeral services in Shiloh, on May 11th, 1915, who stated:

"I must say a word of personal appreciation touching him who has gone from us. I thought a great deal of him. I had come to know him well, as the Professor may come to know the student, as a President of the College may come to know an undergraduate.

"I speak as the one who presides over the College, but also for the class to which he belonged, for the fraternity of which he was a member, for the various organizations of which he was a part, and for the associates with whom he was just now establishing himself in his life work.

"Death coming so suddenly, so unexpectedly, and indeed in so tragic a way, one taken away in the days of his youth, our souls are sorely tried; it seems so premature. Yet, on the other hand, we think of the eternal youth of one going out of this life on into richer life in the prime of his strength, still to do God's will.

* Memoir prepared by F. E. Caldwell, Assoc. M. Am. Soc. C. E.

"I think of this young man so. He was a man, and he was manly. He was reliable, so true and steadfast. He could be sought for a word of good judgment, and he could be sought for a word of intelligent sympathy. He was the sort to become a leader among his fellows, a worker in all good ways of college life. He was the sort to become a leaven in a community of young men, leavening the life of young men with the spirit of devotion.

"I went again and again to him to get the point of view of the student, and never failed to get good judgment, or to find the viewpoint that could realize the other viewpoint, so bringing the best issue.

"It is a fine thing to leave a record like that."

Mr. Bowen's professional career was brief; he was taken in the fullness of his youth, leaving behind the memory of an earnest life, well spent.

Mr. Bowen was elected a Junior of the American Society of Civil Engineers on December 31st, 1913.

WARREN AUSTIN GATES, Jun. Am. Soc. C. E.*

DIED MAY 24TH, 1914.

Warren Austin Gates was born at Coxsackie, N. Y., on March 7th, 1885. He received his early training at the Coxsackie High School, from which he was graduated with a University Scholarship. He was a graduate of the Massachusetts Institute of Technology, in the Class of 1907, and received the degree of Bachelor of Science.

After completing his college course, Mr. Gates worked for a few months with the Buffalo Expanded Metal Company on construction work in Buffalo, N. Y., and in December, 1907, went with the General Fireproofing Company, of Youngstown, Ohio, remaining in the Reinforced Concrete Designing Department of this Company until July, 1908, when his health failed, compelling him to live in the Adirondacks for some time.

In October, 1909, Mr. Gates started to work again, with the firm of Layton Smith and Hawk, Architects, of Oklahoma City, Okla., as Engineer and Superintendent on reinforced concrete construction. He remained with this firm until his death on May 24th, 1914.

When he first went to Oklahoma he held the position of Assistant Engineer, but at the end of a year, he became Chief Engineer in charge of all engineering work in the office. This was a position of trust and responsibility, his work including among other structures, the Houston High School, Houston, Tex., and all preliminary work on the Oklahoma State Capitol, the construction of which was started at about the time of his death.

Mr. Gates was an engineer of exceptional promise, and one who at all times had before him the highest ideals of his Profession. He was a sincere friend and a man who had the complete confidence of all those who were brought in touch with him in his work.

He was married in October, 1911, to Miss Marion W. Sloane, of East Orange, N. J., and is survived by his widow and one daughter.

Mr. Gates was elected a Junior of the American Society of Civil Engineers on June 30th, 1911.

* Memoir prepared by C. J. Paterson, Assoc. M. Am. Soc. C. E.

CURTISS MILLARD, Jun. Am. Soc. C. E.*

DIED FEBRUARY 16TH, 1916.

Curtiss Millard was born at North Egremont, Mass., on March 23d, 1862. He was graduated from the High School at Great Barrington, Mass., in 1880.

He began his engineering work in August, 1881, as a Chainman, etc., on the New York, West Shore and Buffalo Railway, under the late Elmer L. Corthell, President, Am. Soc. C. E., then Chief Engineer of that road. From May, 1884, until May, 1889, he served as Transitman and Assistant Engineer on the New Croton Aqueduct, and from the latter date until August, 1891, he was Engineer for the Contractors for the Bog Brook Dams and the connecting tunnel near Brewster, N. Y. From August, 1891, until March, 1895, Mr. Millard was Roadmaster, Chief Engineer, and Superintendent, successively, of the Charleston, Sumter, and Northern Railroad. From May, 1895, until January, 1896, he was Assistant to the General Freight and Passenger Agent of the Baltimore, Chesapeake and Atlantic Railway, and then, until August, 1896, he was engaged in making examinations and reports relating to water-works and electric lighting plants in Ohio.

In August, 1896, Mr. Millard became Chief Engineer of the St. Louis, Chicago and St. Paul Railway, and from July, 1897, to March, 1900, he was Superintendent of that line. During the last two years of this period he was also Superintendent of the Chicago, Peoria and St. Louis Railway. From March, 1900, until March, 1903, he was General Manager of the Chicago, Peoria, and St. Louis Railway, and from the latter date until February, 1908, President of the Peoria and Pekin Union Railway Company.

About July, 1908, Mr. Millard became Engineer of Maintenance of Way for the Western Division of the Chicago Great Western Railroad, with headquarters at Des Moines, Iowa, which position he retained until January, 1915, when ill health incapacitated him for further work.

It is evident from this record that Curtiss Millard was endowed with exceptional natural ability, since, with only a High School education, he was able to take on responsibilities of a high order. He was devoted to his work, and faithful in its execution. These qualities, combined with a sterling character, made him successful and useful as a man, as an engineer, and as a railroad official.

Robert Ridgway, M. Am. Soc. C. E., writes as follows:

"I was associated with Mr. Millard on the new Croton Aqueduct work between 1884 and 1890, both of us serving in the Engineering Department of the Aqueduct Commission on the location and con-

* Memoir prepared by the Secretary from information on file at the Society House.

struction of the Aqueduct tunnel. This project was the largest municipal engineering undertaking of that day.

"My recollection of him on the work is that of a young man of great physical strength and forceful personality; active and courageous, mentally and physically. He had a direct way of going at things, and no task was too difficult for him to undertake. Well liked and respected by his associates and subordinates, he inspired the confidence of the engineers over him, because of his sound judgment and ability to get quickly results that could be depended upon. His work at that time gave promise of a very successful career."

Mr. Millard was married in 1890 to Miss Caroline Radford, at Yonkers, N. Y. He is survived by his widow and two of his three children.

Mr. Millard was elected a Junior of the American Society of Civil Engineers on April 3d, 1889.

EDWARD WOOLSEY COIT, F. Am. Soc. C. E.*

DIED SEPTEMBER 25TH, 1915.

Edward Woolsey Coit was born on July 27th, 1837, at Plattsburg, N. Y., at which place his father was the Rector of the Protestant Episcopal Church. He was one of six sons, only one of whom, Dr. J. Milnor Coit, of Munich, Germany, now survives. An elder brother, the Rev. Dr. Henry A. Coit, was the well-known and much loved first Headmaster of St. Paul's School, at Concord, N. H. The only academic education Mr. Coit had was that received in his own home, his father having been his instructor.

Mr. Coit began his business career as clerk in a store dealing in general merchandise in Champlain, N. Y. After a residence of about one year at that place he accepted an offer to enter the employ of Morris, Tasker and Company, of Philadelphia, Pa. He remained with that firm all through the period of the Civil War and until 1877, when he resigned to become General Manager of the Reading Iron Works, at Philadelphia, of which Company Mr. John Penn Brock was President at that time. After serving for five years as General Manager, Mr. Coit was elected President of the Company, Mr. Brock having resigned.

In 1892, Mr. Coit became General Manager of the Lake Superior Iron Ore Company, and three years later he moved to St. Louis, Mo., to become Manager of the Branch Works of the National Tube Company in that city. He remained with this Company until 1902, when he moved with his family to California, making his home in Los Angeles.

After a couple of years of much needed rest, Mr. Coit again entered the business world. He became a member of the official staff of the R. H. Herron Oil Wells Supply Company, affiliated with the Oil Wells Supply Company of Pittsburgh, Pa., and remained actively engaged in the affairs of the Company until the day preceding his sudden death on September 25th, 1915.

Mr. Coit was regarded with the highest esteem in both the business and social world. During his long business career he was intimately associated with many of the prominent men of his day, and among his friends and social intimates were Mr. James B. Gowan, a former President of the Philadelphia and Reading Railroad, the late George F. Baer, also a former President of the same road, Messrs. John Penn Brock, Joshua Rhodes, R. T. Crane, and many others. One who knew Mr. Coit most intimately writes as follows concerning his reputation as a man of affairs:

* Memoir prepared by the Rev. Frederick H. Post, Topanga, Cal.

"His reputation was second to none in the iron business, particularly in the manufacture of iron pipe and kindred products. He was an authority whose advice was sought by men of all ranks in that line of business."

The same writer says of his relations to others with whom he came in business contact:

"He believed in giving every man a chance, and advocated fair play in all the various phases of business. He was charitable, believed in young men, and pushed forward a host of them. He had the happy faculty of bringing out the best in men."

Mr. Coit took an interest in every good work, and to many he gave with a liberal hand. If it were possible to describe a man's life and work in one word, that of Mr. Coit could be written in the single, comprehensive word, "gentleman".

While living in Champlain in his youth, Mr. Coit met Miss Caroline Moore whom he afterward married, and who, with two sons and two daughters, survives him.

Mr. Coit was a member of the Protestant Episcopal Church, and was connected with its affairs officially for a number of years. He had been for thirty years a Member of the Manhattan Club of New York City.

Mr. Coit was elected a Fellow of the American Society of Civil Engineers on September 20th, 1872.

"His reputation was second to none in the iron business, particularly in the manufacture of iron pipe and kindred products. He was an authority whose advice was sought by men of all ranks in that line of business."

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Mr. Coit was elected a Fellow of the American Society of Civil Engineers on September 20th, 1871.

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